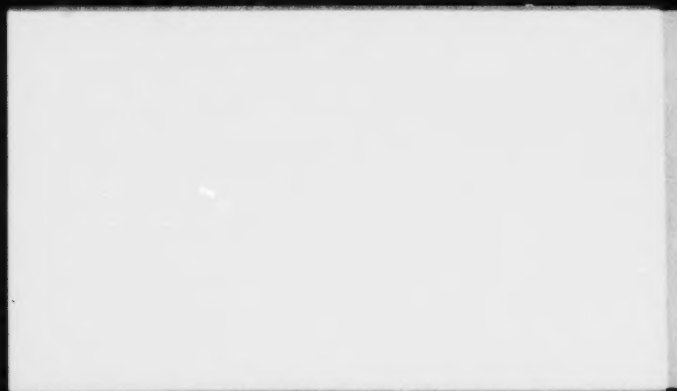




DET NORSKE
VERITAS

EVALUATION OF FRACTURE TOUGHNESS
DATA WITH RESPECT TO RELIABILITY
OF SUBARCTIC AND ARCTIC STRUCTURES

192 pp

Final Report by:

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Calgary, Alberta
Canada

August, 1986

192 pp

Work on the project was conducted under the auspices of
the Canada Centre for Mineral and Energy Technology,
Energy, Mines and Resources Canada.

MICROMEDIA

DSS File No. 06SQ.23440-5-9191

DSS Contract Serial No. 1SQ85-00206

CANMET Project No. 708801

CANMET Scientific Authority: Dr. W.R. Tyson

ABSTRACT

This study, which has been funded by CANMET and DnV, describes and demonstrates the application of probabilistic methods for the strength assessment of tubular joints of offshore structures. The low (arctic) temperatures can initiate unstable fracture from cracks developed during fabrication or the design life of structures. The probabilistic analysis was based on two failure mechanisms:

- (i) fatigue crack propagation;
- and (ii) unstable fracture.

The DnV program PROBAN has been used for the analyses reported here. The study evaluates the sensitivity of failure to such basic factors as fracture toughness of the base plate and the HAZ, initial flaw size, crack growth parameters, hot spot stress range and nominal stress.

The applicability of a Level 2 first order reliability method has been demonstrated on selected configurations of T-, Y- and K- tubular joints. The results obtained from such an approach provide the basis for extending the applicability of standard and high strength steels to low temperature service.

SOMMAIRE

Ce rapport contient les résultats d'une étude qui fut subventionnée à parts égales par Det norske Veritas (DnV) et CANMET. Le but de l'étude est d'appliquer certains modèles aléatoires sur l'évaluation de la résistance de joints tubulaires dans des structures offshore. Les températures dans l'Arctique basses peuvent initier l'extension instable de fissures développées durant la fabrication ou la vie de la structure.

Par conséquent, le modèle aléatoire se base sur les deux mécanismes de défaut suivants:

- (1) l'extension de fissures causée par la fatigue.
- (2) la fracture instable.

L'analyse s'est produite au moyen du programme PROBAN qui appartient à DnV. La sensibilité des défauts à certains facteurs fondamentaux tels que la tenacité de la plaque de base, la longueur initiale de la fissure, les paramètres qui influencent la croissance de la fissure, et finalement les contraintes nominales et celles de la région chauffée fut examinée.

On démontre que les méthodes de fiabilité du premier ordre au niveau 2 sont applicables à des joints tubulaires en forme T, Y et K. Les résultats peuvent servir à étendre l'usage d'aciers de qualité standard et exceptionnelle envers des applications à basses températures.

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ACKNOWLEDGEMENTS

The assistance of J. Lereim, H.O. Madsen, L. Tvedt and R. Skjong of Det norske Veritas, Oslo, is highly appreciated.

1. INTRODUCTION

Structural reliability methods evolved in the past 50 years. However, after a slow beginning, the last decade witnessed an accelerated interest of many sectors of industry in reliability-based structural design. Several codes based on reliability methods were developed and implemented in short succession (CSA, 1974; NKB, 1977; OHBDC, 1983) and the procedures were well documented in guideline reports (CEB, 1976; CIRIA, 1977; CSA, 1981). The codes were calibrated using reliability methods and partial safety factors were introduced, maintaining simplicity in the use of the design criteria (Røren, Madsen and Lereim, 1986). For offshore structures, the partial safety factor format was introduced in 1977 (DnV, 1977).

The large amount of information scattered through professional journals was gathered in monographs in this area (Thoft-Christensen and Baker, 1982; Madsen, Krenk and Lind, 1986). At the beginning, the reliability methods were developed for simple structural elements. At present, using powerful computer techniques, reliability methods applicable to large structures and systems are being developed and implemented.

Various limit states (buckling, yielding, fatigue, unstable fracture) leading to the failure of structures can be identified and the corresponding probabilities of failure evaluated. However, it must be emphasized that the probabilities of failure are not properties of the structures, but change as more information becomes available. Although the computed probabilities of failure cannot be interpreted as expected failure frequencies, they serve well for comparing reliabilities of different designs, different loading conditions, different material properties, etc. They are also very efficient in identifying important sources of uncertainty by comparing sensitivity of the given failure mode to various basic quantities (Madsen, Skjong and Mohtaderi Zadeh, 1986). The probabilities of failure, however, become more meaningful when calibrated for a particular structure. For example, if a structure undergoes a loading regime (pipe laying, pressure testing, etc.) without failure, then the correspondent probability of failure can be used for calibration. Any other loading regime (or a change of loading condition in service) can be evaluated and assessed based on that experience. If the same probabilistic approach has been used in both cases and the calculated probability of failure for the new loading conditions is less than that calculated for the regime which the structure survived, there is a good chance that the

new loading conditions would not lead to its structural failure.

The structural reliability methods are gaining a wide recognition in the offshore and marine industry, and some cases of application of these methods in the North Sea are presented in **Appendix A**.

The objective of this project was to use the structural reliability analysis as a tool and assess the effect of the variability in fracture toughness on the probability of failure of selected tubular joints. The failure modes include those of fatigue crack propagation and unstable fracture (e.g. Tomin, 1983). The results show how this effect (sensitivity) changes with time (service period) and with variation of other parameters.

2. STRUCTURAL RELIABILITY ANALYSIS

2.1 Structural Reliability and Probability of Failure

The structural reliability can be defined as a "measured chance" (probability) P_R , that a structure will withstand without failure, the loads to which it is subjected during some specified time (often the projected or design life). The failure probability, P_f , is then defined as a complement of the structural reliability, so that

$$P_f = 1 - P_R. \quad (2-1)$$

The quantities (variables) which affect the probability of failure form a vector of basic quantities $\underline{Z} = (Z_1, Z_2, \dots, Z_n)$ where $\underline{Z}$ is a random vector in an n-dimensional vector space. It is common to use lower case letters ($\underline{z}$) to denote particular realisations (vectors) in the basic quantity space ($\underline{Z}$). Any limit (failure) state can be defined using a failure function $g(\underline{Z})$. Then the failure surface, corresponding to a given limit state, can be defined by $g(\underline{z}) = 0$, Figure 2-1. The failure surface then divides the space into a subspace of no failure states ($g > 0$) and its complement ($g < 0$) of failure states.

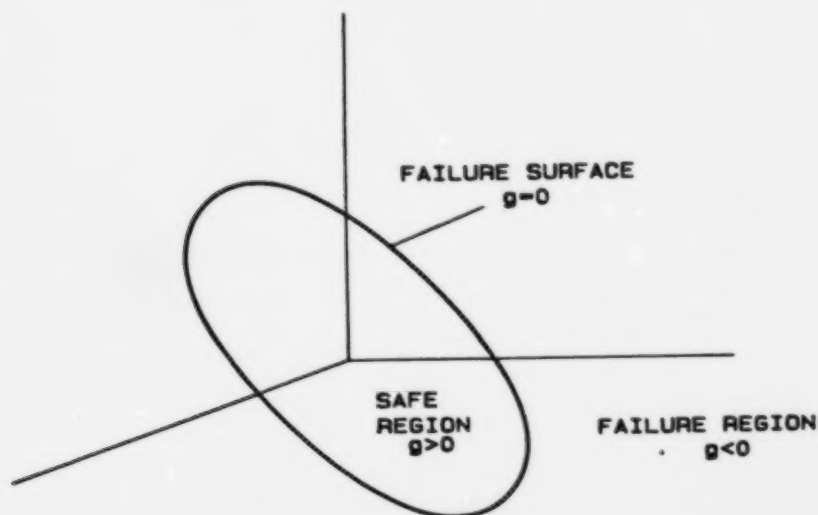


Figure 2-1. Failure Surface

Then, the reliability P_R of a structure is the probability that the realization of $\underline{Z}$ falls in the safe region. Conversely, the probability of failure P_f is the probability that the realization of $\underline{Z}$ falls in the failure region.

The basic quantities can be divided in two groups, those affecting capacity (C), and those affecting demand (D) with corresponding resultant distributions, **Figure 2-2**. The probability of failure then corresponds to the overlapped region and can be expressed in terms of the probability density function of demand (f_D) and the cumulative distribution function of capacity (F_C) as:

$$P_f = \int_0^{\infty} F_C(x) f_D(x) dx \quad (2-2)$$

It is convenient to use the above approach and define a new random quantity, $M = C - D$, as the margin of safety. The probability of failure is then simply

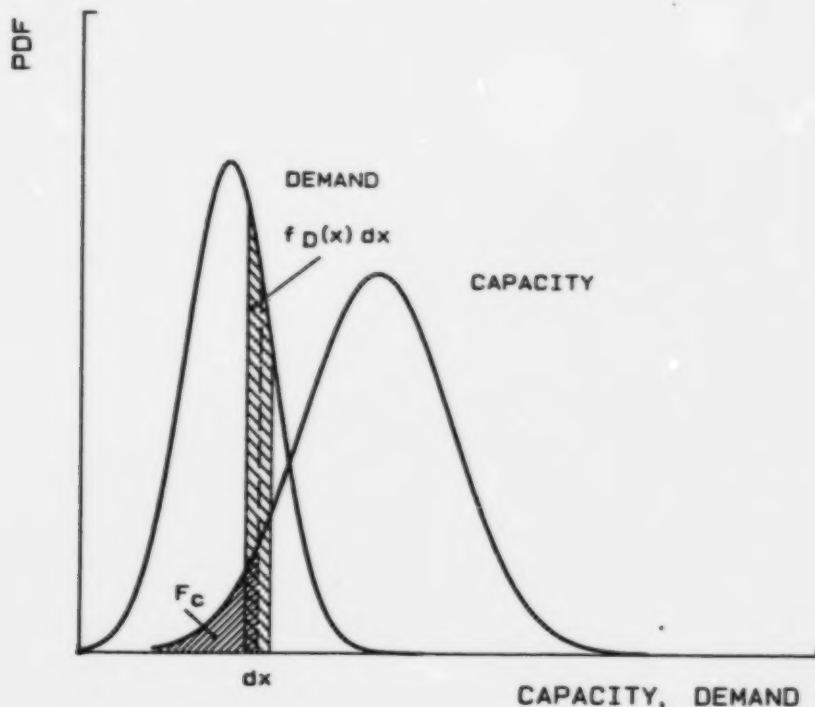


Figure 2-2. Overlapped Region of Capacity and Demand

$$P_f = P(M < 0), \quad (2-3)$$

so that the margin of safety coincides with the previously introduced failure function, i.e.

$$M(\underline{Z}) = g(\underline{Z}).$$

The probability of failure is then given by the lower tail of the margin of safety cutoff at $M = 0$, Figure 2-3.

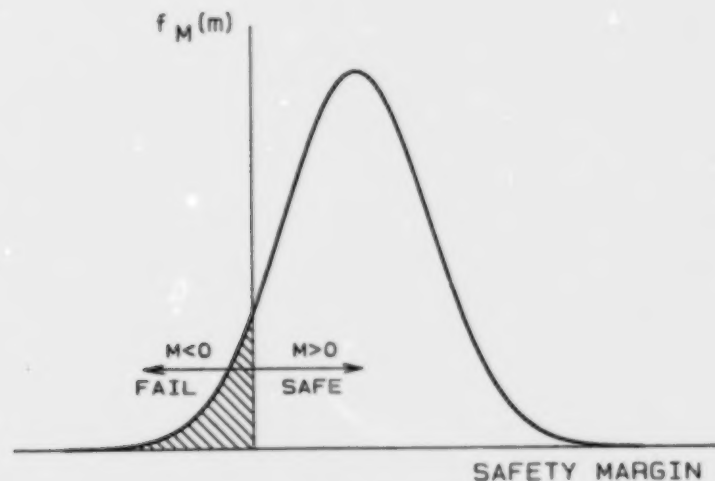


Figure 2-3: Probability Density Function of the Safety Margin

2.2 Levels of Reliability Methods

In using the reliability-based methods, various degrees of simplification led to the definition of several levels of reliability analysis (CIRIA, 1977; Røren, Madsen and Lereim, 1986).

Level 1:

The reliability of structural elements is based on characteristic values (such as means and variances) of the basic quantities and expressed in terms of partial safety factors. In general, the design value, Z_i^* of the quantity Z_i is given by:

$$Z_i^* = m_i - \alpha_i \beta \sigma_i = m_i (1 - \alpha_i \beta V_i)$$

$$= \left(\frac{1 - \alpha_i \beta V_i}{1 + k_i V_i} \right) Z_{ki} = \gamma_i Z_{ki}, \quad (2-4)$$

where

- m_i is the mean value of Z_i ;
- V_i is the coefficient of variation of Z_i ;
- σ_i is the standard deviation of Z_i ;
- Z_{ki} is the characteristic value of Z_i ;
- k_i is a coefficient depending on the probability level p associated with the characteristic value of Z_i , and the nature of the probability distribution of Z_i ;
- α_i is a coefficient depending on the form of the failure boundary;
- γ_i is the desired partial safety factor; and
- β is the reliability index (see Section 2.3).

For example, ISO 2394 (1973) quotes seven partial factors to be applied to the loads or to the structural strength. The safety format of the DnV rules (1977) simplified this approach introducing two partial safety factors, one for loads, γ_f and one for material, γ_m . Then, the typical inequality regarding safety by applying the partial safety factor format is:

$$C(R_c / \gamma_m) > D (L_c \gamma_f)$$

where R_c is the characteristic resistance;

L_c is the characteristic load; and

C and D denote capacity and demand respectively.

Level 2:

These methods use computational algorithms idealizing the failure surface at a selected point (design point) when calculating the failure probability of a structure. Often, these methods are based on Hasofer-Lind reliability index (Hasofer, Lind, 1974). This method was used in this project and will be discussed in more detail in the following section.

Level 3:

These reliability methods employ full joint probability distribution functions of basic quantities, making the analysis computationally much more demanding. Level 3 methods evaluate the probability content of the entire failure region. The reliability levels are commonly based on agreed failure probabilities, interpreted in the sense of relative frequencies. The Level 3 methods are currently being applied in the new generation of reliability software.

Level 4:

These methods make the reliability analysis a part of an overall engineering economic model under uncertainty, considering the cost benefits of construction, maintenance and repair, consequences of failure, etc. These methods are being developed at present.

2.3 First-Order, Level 2 Reliability Method

Rackwitz (1976) grouped the Level 2 reliability methods into three classes:

1. Mean value first-order second-moment methods;
2. Advanced first-order second-moment methods; and
3. Approximate full distribution methods.

These three distinct classes are discussed in detail in CIRIA (1977). The Level 2 reliability analysis, implemented in PROBAN (PROBability ANalysis computer program; Tvedt, 1983) belongs to the third class of Rackwitz, which is a generalized class 2 for nonnormal distributions of basic quantities. Only the basics of the algorithm implemented in PROBAN used for this project will be described.

Cornell (1969) introduced the reliability index (safety index), β as:

$$\beta = \frac{\mu_M}{\sigma_M},$$

where μ_M is the mean and σ_M is the standard deviation of the safety margin, **Figure 2-4**. Thus, the number of standard deviations, by which the mean of the safety margin exceeds zero, became a convenient measure of reliability.

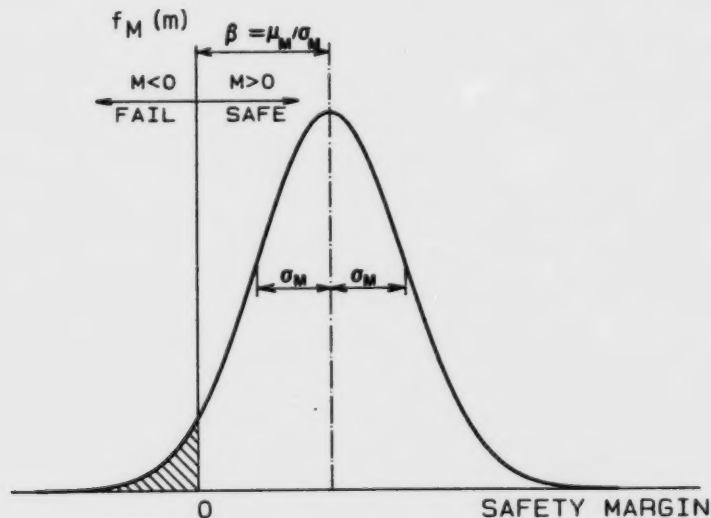


Figure 2-4. Illustration of the Reliability Index β

The concept of the reliability index can be generalised and $\beta(\underline{z})$ can be interpreted as a measure of the distance from the origin to the failure surface in the n-dimensional space of the vector $\underline{z}$. This is illustrated in Figure 2-5. As can be seen from this figure, the reliability index proposed by Hasofer and Lind (1974) β_{HL} corresponds to the minimum distance from the origin to the failure surface. The point on the failure surface defining this minimum distance is named the design point. When M is normally distributed (C and D are normally distributed and uncorrelated), then the probability of failure is simply:

$$P_f = P(M < 0) = \Phi\left(\frac{0 - \mu_M}{\sigma_M}\right) = \Phi(-\beta),$$

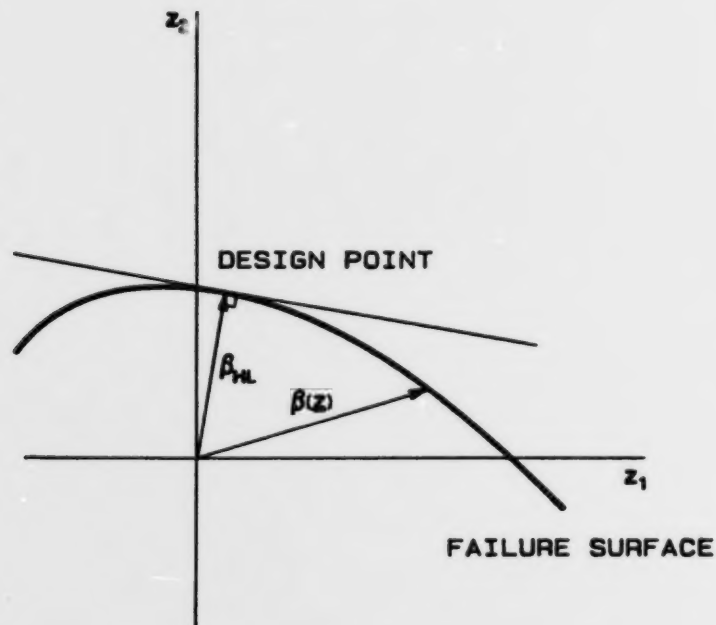


Figure 2-5. Geometrical Interpretation of the Reliability Index

When the basic quantities are not normally distributed and possibly correlated, the first step in the reliability analysis is the one-to-one transformation of basic quantities $\underline{Z}$ into a set of independent and standardized, normally distributed variables $\underline{U}$, Figure 2-6. It is convenient to define a sensitivity, α_i , of the reliability index at the design point ($\underline{u}^*$) with respect to the basic quantity (u_i) as:

$$\alpha_i = \frac{u_i^*}{\left(\sum_{j=1}^n u_j^{*2} \right)^{1/2}} = \frac{u_i^*}{\beta_{HL}} \quad (2-7)$$

where β_{HL} is the Hasofer-Lind reliability index.

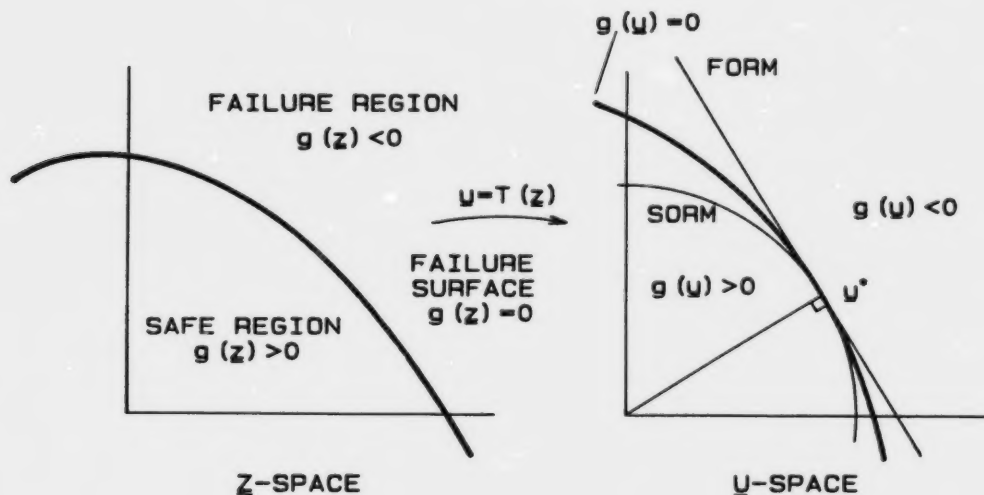


Figure 2.6. Transformation from $\underline{Z}$ -Space to $\underline{U}$ -Space

The sensitivity factors α_i are the directional cosines at the design point, Figure 2-7, so that:

$$\sum_{i=1}^n \alpha_i^2 = 1.$$

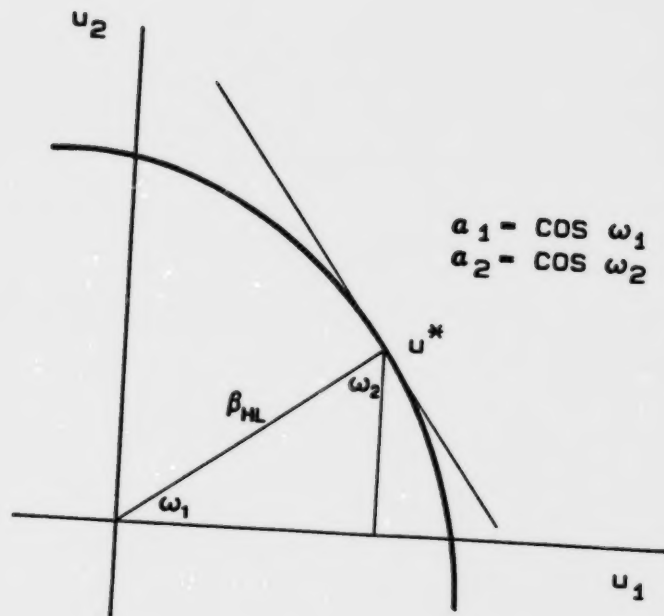


Figure 2-7. Illustration of Sensitivity Factors

The First-Order Reliability Method (FORM) uses a hyperplane approximation of the failure surface at the design point, Figure 2-6. Correspondingly, the Second-Order Reliability Method (SORM) uses a quadratic approximation of the failure surface. It can be shown that FORM provides a good approximation for most engineering problems, since the major contribution to the failure probability stems from the shaded area in Figure 2-8. This is attributed to the fact that the probability density function of the basic quantities decreases as $\exp(-r^2/2)$ with the distance r from the origin (Madsen, Krenk, Lind, 1986). The use of FORM reduces considerably the amount of computational work.

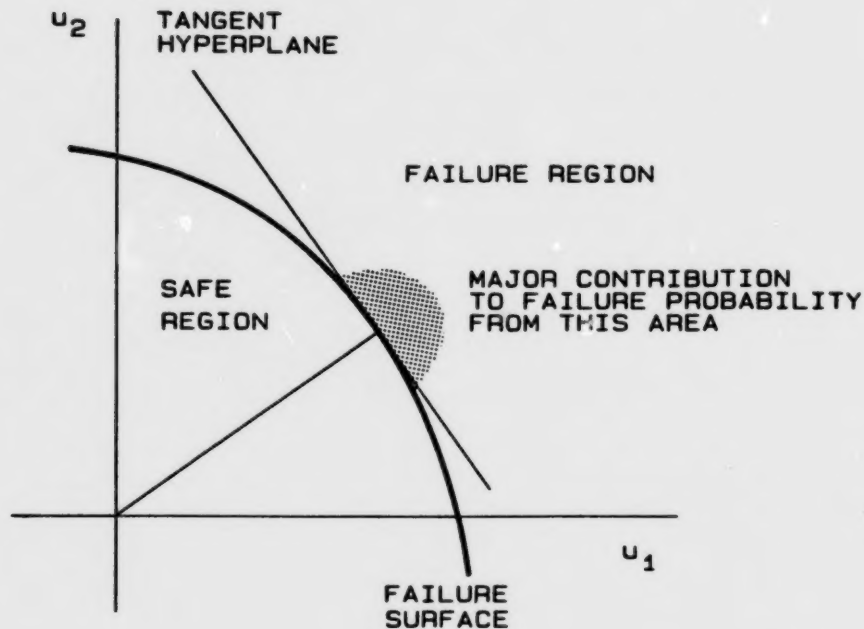


Figure 2-8. FORM Hyperplane Approximation

2.4 Computer Program PROBAN

The computer program PROBAN was developed by DnV with sponsorship by CONOCO and U.S. Coast Guard (Tvedt, 1983a, 1983b). In PROBAN, the following algorithm was implemented (Rackwitz and Fiessler, 1978):

- Step 1: Compute the mean and standard deviation of the equivalent normal distributions at the mean $\underline{z}_0$ of the basic distributions. This defines a linear transformation into the iteration space $\underline{u}_0$ that is fixed until the next transformation.
- Step 2: Compute the u-space gradient to the limit state (failure) function at the mean of the $\underline{z}$ -space distributions. Iterate until an intersection $\underline{u}_0(\underline{z}_1)$ with the surface is found. Set $i = 1$.

Step 3: Compute the mean and standard deviation of the equivalent normal distributions at $\underline{z}_i$. This defines the transformation into the $\underline{u}_i$ space for the next iteration. Compute $\beta_i = \text{sign}(g(\underline{z}_0)) |\underline{u}_i(\underline{z}_i)|$ in this space.

Step 4: Compute the gradient $\nabla g(\underline{u}_i(\underline{z}_i))$. Define the plane given by $\underline{u}_i(\underline{z}_i)$ and the gradient. Iterate in this plane along the circle with center at the origin and radius $|\beta_i|$ until a second intersection α_s with the surface is found, i.e. solve

$$\underline{u}_i(\alpha) = \underline{u}_i(\underline{z}_i) \cos(\alpha) + \underline{v}_i \sin(\alpha),$$

where $\underline{v}_i$ is the orthogonal projection of $\nabla g(\underline{u}_i(\underline{z}_i))$ onto $\underline{u}_i(\underline{z}_i)$ scaled to the length $|\underline{v}_i| = |\underline{u}_i(\underline{z}_i)|$.

Step 5: Bisect the intersection angle α_s and iterate along the direction $\underline{u}_i(\alpha_s/2)$ until a new intersection with the surface is found. This defines the next iterate $i+1$.

Step 6: Stop-criteria:

$$1: \quad |\beta_{i+1} - \beta_i| \leq 0.001$$

$$\text{Arccos}\left(\frac{\underline{u}_i \cdot \underline{u}_{i+1}}{|\underline{u}_i| |\underline{u}_{i+1}|}\right) \leq 0.01$$

$$2: \quad 2\beta_i (1 - \cos\alpha) \leq 0.001$$

$$2 |\alpha| \leq 0.01$$

where

$$\alpha = \text{Arccos}\left(\frac{\underline{u}_i \cdot \nabla g(\underline{u}_i)}{|\underline{u}_i| |\nabla g(\underline{u}_i)|} \text{sign}(g(\underline{z}_0))\right).$$

The first criterion must be fulfilled two times in succession, or the second criterion must be fulfilled once. If one of the stop criteria are fulfilled, jump to 7 else jump to 3.

Step 7: Output results:

The iteration procedure applied in the program is a method of steepest distance descent on the failure boundary, Figure 2-9.

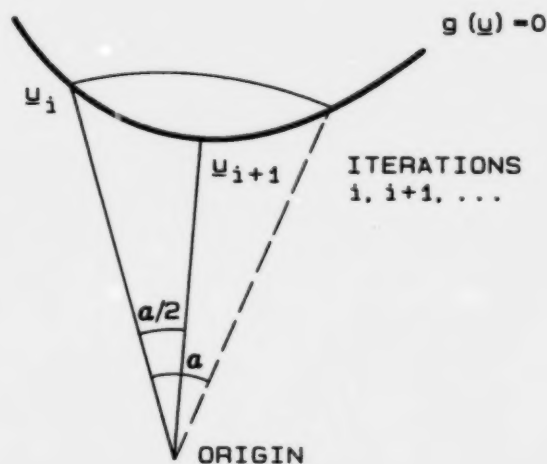


Figure 2-9. Iteration Along the Failure Surface in the Standardized Normal Space

Presently, the following distributions of basic quantities are implemented in PROBAN:

- normal
- lognormal
- Weibull
- Gumbel
- Longuet-Higgins
- Nolte
- Generalized Gamma

The basic quantities can be independent or correlated. The number of random quantities is limited to 50 which surpasses the needs of most engineering problems. The user defines the failure surface (limit state) function $g(\underline{Z})$ in the user subroutine LIMST which is then compiled PROBAN, Figure 2-10.

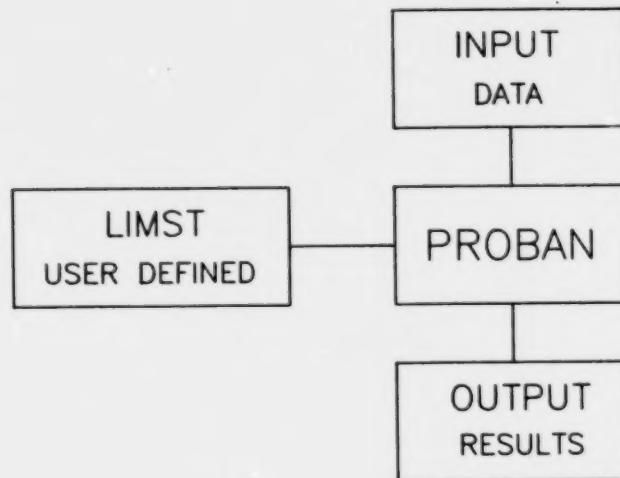


Figure 2-10. Schematics of Running PROBAN

A typical output of a PROBAN analysis is shown in Appendix B.

2.5 Targeted Probabilities of Structural Failure

Even though the probabilities of failure obtained from the reliability analysis cannot be interpreted in terms of failure frequencies, it is still of interest to discuss the targeted (acceptable) occurrence of structural failures. Such targets of the probability of failure are implemented implicitly or explicitly in some structural codes (see Introduction), and are based on natural disasters, risks involved in various human activities, and the foreseeable consequences of structural failures. The consequence (a loss) can be divided into four categories: economic loss, damage to the

environment, injury or death. The risk of death for various human activities is shown in Table 2-1 (Madsen, Krenk and Lind, 1986).

Table 2-1. Risk of Death (Source, CIRIA, 1977)

Exposure	Risk of Death per Hour per 10 ⁶ Persons Exposed	Hours of Exposure per Persons Exposed per Year	Risk of Death per 10 ⁶ Exposed Persons per Year	Ratio of Wounded to Number of Deaths
Mountain climbing (international)	2700	100	27	
Trawl fishing (deep sea, 1958-1972)	59	2900	17	is in the t-spot"
Flying (crew)	120	1000	12	<<1
Coal mining	21	1600	3.3	
Automobile travel	56	400	2.2	20
Construction	7.7	2200	1.7	450 e brace usually
Flying (passengers)	120	100	1.2	<<1
Home accidents	2.1	5500	1.1	The derable
Factory work	2	2000	0.4	failure
Building fires	0.15	5500	0.08	5
Structural failure	0.002	5500	0.001	6

For example, there are no internationally accepted target values for probability of failure in shipping, even though 10^{-4} is often regarded as acceptable (Lindeman, Odland and Strengenhagen, 1977). Various authors have proposed formulae for the target values, e.g. Flint and Baker (1977) suggest:

$$P_f = 10^{-4} \frac{K_s}{N_r} N_d, \quad (2-9)$$

where N_d = design life in years;
 N_r = number of people at risk; and
 K_s = social criterion factor, varying from 5 (hazardous activities) to 0.5 (structures where sanctuary is provided).

If we assign, for example, $K_s = 3$ for Arctic shipping, $N_d = 20$ years and $N_r = 20$ people, the target value for failure probability from Eq. (2-9) is $P_f = 3 \times 10^{-4}$. In Figure 2-11, the risk of various events in terms of fatalities is shown (Boe, Joergensen, Mathiesen, and Røren, 1974).

Offshore steel structures, designed to meet recognized codes, generally have a theoretical probability of failure in the order of 10^{-4} to 10^{-6} . Concrete structures usually fall in the lower part of that range. DnV Rules (1977) for steel members and for the plasticity based analysis (partial material factor of 1.3) target on a probability of failure of about 10^{-6} (Fjeld, 1976). It is argued that targeted risk levels for offshore structures should not be much higher than those accepted for onshore industry.

The failure probabilities can be classified in the following four categories (Fjeld, Andersen and Myklatun, 1978):

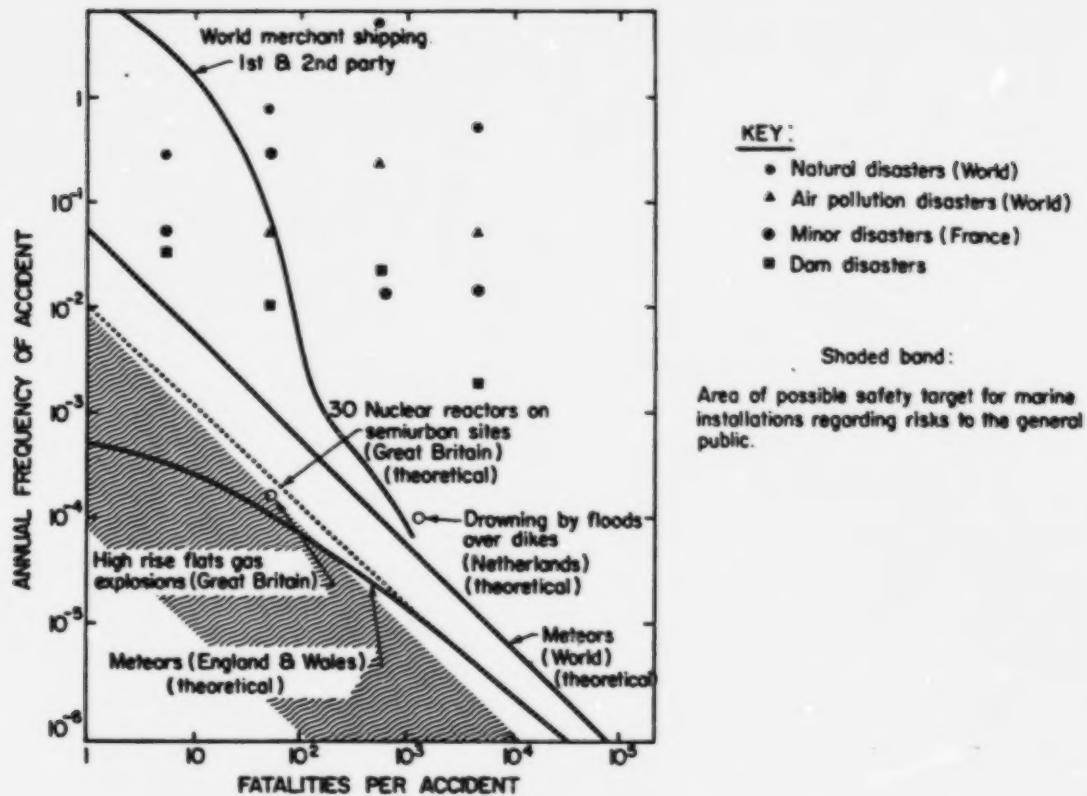


Figure 2-11. Compiled Data on Accident Frequency and Related Consequences (Fatalities)

	P_f	$< 10^{-6}$	negligible
$10^{-6} <$	P_f	$< 10^{-4}$	low
$10^{-4} <$	P_f	$< 10^{-2}$	medium
	P_f	$> 10^{-2}$	high

The probabilities of failure targets, as implemented in various structural codes, are shown in Table 2-2 (Fjeld, 1976):

Table 2-2. Probability of Failure Targets in Some Codes

Code	Prob. of failure during life	Annual prob. of failure
IMPLICIT PROBABILITIES		
AISC 1969	$3 \cdot 10^{-3}$	10^{-4}
ACI 318 '71	$9 \cdot 10^{-5}$	$3 \cdot 10^{-6}$
Canadian structural codes	10^{-3}	$3 \cdot 10^{-5}$
EXPLICIT PROBABILITIES		
ECCS general		10^{-5}
dangerous consequences		10^{-6}
CIRIA report 63	Onshore industry	$10^{-6} - 10^{-7}$
	Offshore industry	$10^{-4} - 10^{-6}$

3. TUBULAR JOINTS IN OFFSHORE STRUCTURES

3.1 Joint Classification

Tubular joints are basic structural elements of fixed offshore structures (jackets). All aspects of the design of tubular joints is extensively covered in the UEG (1985) monograph. Tubular joints are usually classed symbolically by using capital letters T, Y, K, X, which they resemble, and D for "double", Figure 3-1.

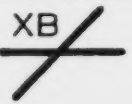
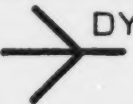
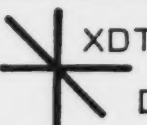
SINGLE JOINTS	DOUBLE JOINTS
 T	 DT
 Y	 XB  DY
 K	 DK
 YT	 XDT  DYDT
 KT	 DKDT

Figure 3-1. Preferred Notation for Joint Configurations (UEG, 1985)

The stress concentration factors (SCF) for tubular joints are usually expressed in terms of the following non-dimensional parameters, Figure 3-2:

Chord length parameter α

α is defined as the ratio of chord length L to chord radius $D/2$ (i.e. $2 L/D$) and gives an indication of chord beam bending characteristics.

Diameter ratio β

β is the ratio of the brace diameter to the chord diameter (i.e. d/D) and describes the compactness of the joint.

Chord thinness ratio γ

γ is the ratio of the chord radius to the chord wall thickness (i.e. $D/2T$) and gives an indication of the thinness and radial stiffness of the chord.

Wall thickness ratio τ

τ is the ratio of the wall thickness of the brace to that of the chord, it is a measure of the likelihood that the chord wall will fail before the brace cross section fractures.

Gap parameter ζ

ζ is defined as the ratio of the gap to chord diameter. This parameter describes the proximity of other brace members to the subject brace member for joints with more than one brace.

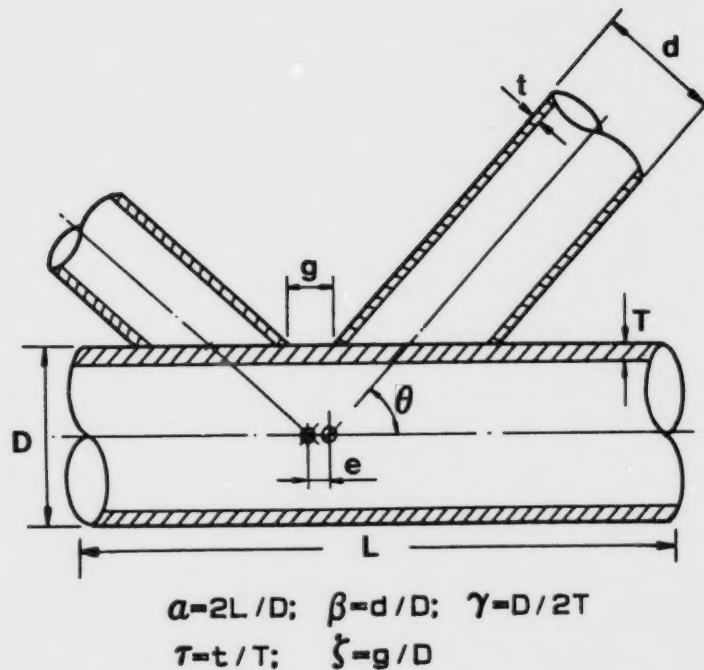


Figure 3-2. Geometric Notation

Six offshore structures in the North Sea have been surveyed for the occurrence of various configurations of tubular joints (UEG, 1985) and these results were respected in selecting the joint geometries for this study.

3.2 Stress Concentration Factors

The stress varies considerably along the brace-chord intersection of a tubular joint, Figure 3-3, and the location of the maximum stress ("hot-spot" stress) depends on the joint geometry and on the type of loading.

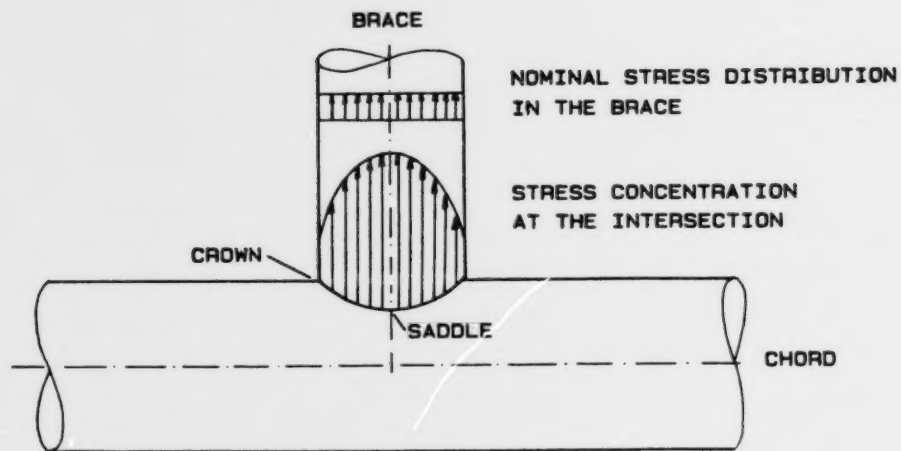


Figure 3-3. Stress Concentration at the Intersection

See UEG (1985) for various definitions of "hot-spot" stress.

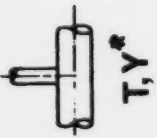
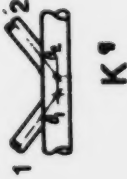
The three types of loading (transfer of load from brace to chord) are:

- axial loading;
- in-plane bending (IPB); and
- out-of-plane bending (OPB).

To keep the number of cases for this project manageable, only axial loading was considered.

Based on various methods (analytical, numerical, experimental) used to analyze the "hot-spot" stresses of tubular joints, various formulae have been recommended (e.g. Kuang, Patvin and Leick, 1977; Wordsworth and Smedley, 1978; Gibstein, 1978; or Dijkstra, 1981). Based on a review and comparison of various "hot-spot stress" formulae, the recommended formulae for axial loading (UEG, 1985) are given in Table 3-1.

TABLE 3-1
Recommended Characteristic Formulae
for Axially Loaded Simple Joints (UEG, 1985)

JOINT TYPE	CHORD SADDLE SCF §	CHORD CROWN SCF §	VALIDITY RANGE
 T, Y*	$\gamma \tau \beta (6.78 - 6.428 \beta^{1/2})$ $\times \sin^{1.7+0.7\beta^3} \theta (Q_B Q_Y)^{1/2}$	$(X'_C + X'_O X'^{11}_C) / Q'_Y$ where $X'_C = (0.7 + 1.37 \gamma^{1/2} \tau (1 - \beta)) (2 \sin^{1/2} \theta - \sin^3 \theta)$ $X'_O = \frac{\tau (2 \gamma \beta - \tau) (\alpha/2 - \beta / \sin \theta) \sin \theta}{2 \gamma - 3}$ $X'^{11}_C = 1.03 + \frac{30 \tau^{1.5} (1.2 - \beta) (\cos^4 \theta + 0.15)}{\gamma}$	$2.5 < \alpha < 40$ $0.13 \leq \beta \leq 1.00$ $12.0 \leq \gamma \leq 40.0$ $0.25 \leq \tau \leq 1.00$ $30^\circ \leq \theta \leq 90^\circ$
 K*	$\gamma \tau \beta (6.78 - 6.428 \beta^{1/2})$ $\times \left(\sin^{1.7+0.7\beta^3} \theta_1 \right)$ $+ (\sin^{1.7+0.7\beta^3} \theta_2)$ $\times (0.012 \gamma)^{1/2} (3 \tau + 0.4) \left(\frac{\sin \theta_1}{\sin \theta_2} \right)^{1.8}$ $\times (Q'_B Q'_Y)^{1/2}$	$1.1 \tau^{0.65} \tau \left(\frac{\sin \theta_1}{\sin \theta_2} \right) (2 \tau)^{0.05 / \beta} (1.5 \beta^{0.25} - \beta^2) Q'_Y$	$0.13 \leq \beta \leq 1.00$ $8.0 \leq \gamma \leq 40.0$ $0.25 \leq \tau \leq 1.00$ $30^\circ \leq \theta \leq 90^\circ$ $\theta_1 \geq \theta_2$

Note: $Q'_B = 1$ for $\beta < 0.6$ and $0.3 / \beta (1 - 0.833 \beta)$ for $\beta > 0.6$; $Q'_Y = 1$ for $\gamma < 20.0$ and $480 / \gamma (40 - 0.833 \gamma)$ for $20 < \gamma < 40.0$. The SCFs are limited to a minimum of 1.5. Lower values are allowed if justified by experimental or numerical evidence.

* For T/Y joints, the chord length can be taken as the distance between points of contraflexure on the chord.

§ For brace SCF use $SCF_b = 1 + 0.63 SCF_c$.

* For K joints where $\theta_1 \geq \theta_2$, formulae for SCFs refer to brace 1 and are based on nominal stress in brace 1.

3.3 Failure Modes of Tubular Joints

A safe design of tubular joints must take account of all foreseeable failure modes which could occur during the service of the structure. The following failure modes can be readily identified (UEG, 1985):

- plastic failure of the chord;
- cracking and gross separation of chord from brace;
- cracking of brace;
- shear failure of the chord between adjacent bracings; and
- lamellar tearing.

Different failure mechanisms are observed for compressive and tension loads in the brace. In tension, as the load increases, a crack may be initiated at the "hot-spot", which eventually leads to separation of brace from chord. In compression, a plastic buckling and bending of the chord wall usually take place.

The in-plane bending moment loading leads usually to fracture through the chord wall on the tension side of the brace and to plastic bending and buckling of the chord wall on the compression side.

The out-of-plane bending leads to local buckling of the chord wall near the brace saddle, thus reducing its stiffness. Fracture on the tension side of the brace usually takes place after excessive plastic deformation.

The loading of offshore structures (waves, winds) is highly stochastic. The number of cycles during the lifetime of such structures may lead to considerable fatigue crack growth with subsequent unstable fracture. This type of failure mechanism is considered in detail in this project.

4. PROBABILISTIC MODEL

4.1 General Comments

The probabilistic model used for the study is described in the following sections. First, the geometries of the structural elements (tubular joints) are defined. These geometries define in turn the corresponding stress concentration factors needed for the analysis. Further, a failure model based on fracture criteria is outlined and finally, the basic quantities for this model are identified and quantified.

4.2 Joint Configurations

The T-, Y- and K-joints (see Section 3.1) were selected for the analysis. The geometries of the joints (non-dimensional parameters) correspond to those most frequently used in existing offshore structures (UEG, 1985).

T-Joint: A T-joint configuration is the one most extensively investigated (e.g. Fraser and Kearns, 1982). A major eight-leg structure of cluster pile configuration in the North Sea, constructed in 1979 for 146 m water depth (UEG, 1985) is made up by 72.5% of T-joints. About 62.5% of the T-joint population have the β ratios in the range 0.4 to 0.6 and γ ratios in the range 10 to 20. The following configuration was chosen for this study, **Figure 4-1:**

$$\beta = \frac{d}{D} = 0.60,$$

$$\gamma = \frac{D}{2T} = 18.75,$$

$$\tau = \frac{t}{T} = 0.5, \text{ and}$$

$$\alpha = \frac{2L}{D} = 10.$$

The T-joint can be regarded as a special case of the Y-joint for $\theta = 90^\circ$ (see Figure 3-2 and Table 3-1).

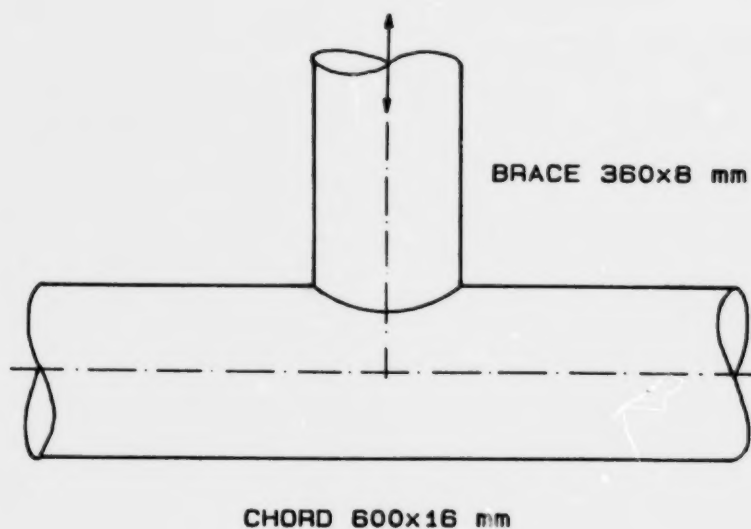


Figure 4-1: T-Joint Configuration

Y-Joint: The simple Y-joint configuration usually amounts to less than 10% of the total tubular joint population. The same dimensions as for the T-joint have been used with the angle $\theta = 45^\circ$, Figure 4-2.

K-Joint: The K-joint configurations are highly represented in the north Sea jackets. The K-braced jacket structures use almost entirely K-joints (simple or multi-planar). For example, in the design of a major eight-leg structure of skirt pile configuration in the North Sea, constructed in 1978 for 143 m depth, simple K-joints made 16.9% of the total (1,014) joint population (UEG, 1985). A symmetrical K-joint ($\theta_1 = \theta_2 = 45^\circ$), with the same dimensions of the braces and the chord as for T- and Y-joints, is assumed. The gap of $g = 30$ mm, Figure 4-3, gives $\zeta = g/D = 60/600 = 0.1$.

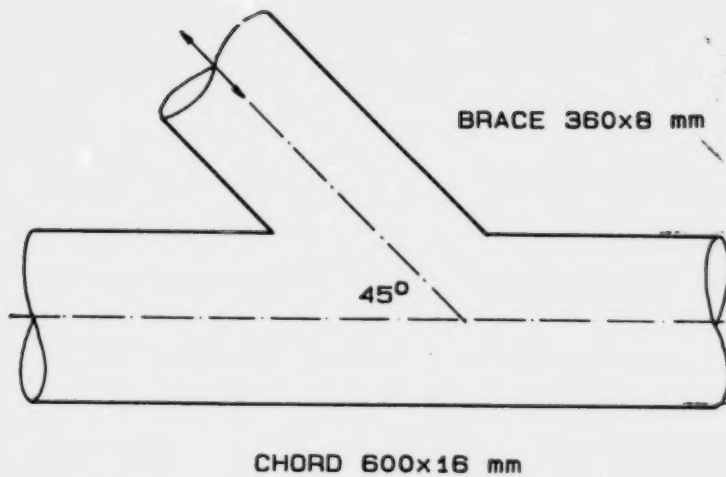


Figure 4-2. Y-Joint Configuration

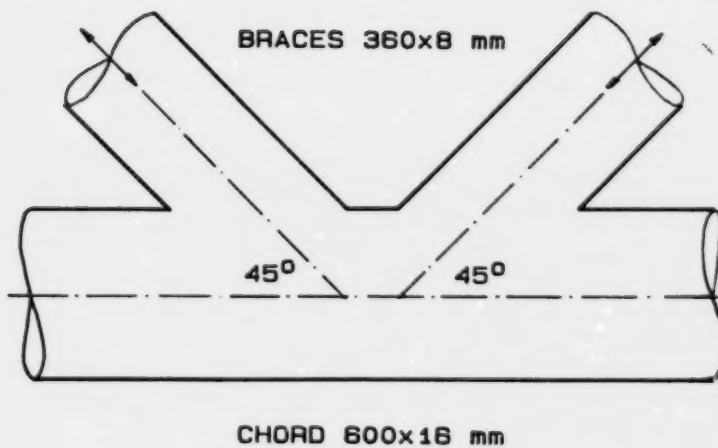


Figure 4-3. K-Joint Configuration

Both braces interact in the joint due to the following two effects:

1. the stiffening effect introduced by the second brace; and
2. the interaction of stress fields of the neighbouring connections.

The location and magnitude of the "hot-spot" stress will be highly dependent upon the geometry of the joint expressed through the non-dimensional parameters. For a balanced axial loading of the braces (tension in one brace balanced by compression of the same magnitude in the other brace), the SCF is reduced when compared with an axially-loaded Y-joint of the same dimensions. However, when both braces carry loads of equal magnitude and sign, the result will be a higher SCF. This case will be considered here.

The SCF's corresponding to the T-, Y- and K-joints considered are given in Table 4-1. Note that $\sigma_{HS} = SCF \sigma$, where σ is the nominal stress in the brace.

Table 4-1

Stress Concentration Factors for Axial Loading

JOINT	CHORD SADDLE	CHORD CROWN
T	10.2	3.4
Y	5.4	3.6
K	8.0	3.1

The values of the SCF's indicate that the fatigue cracks would initiate and propagate at higher rates at chord saddle rather than at chord crown, assuming the probability of the initial defect size along the chord-brace intersection is constant. The crack propagation and unstable fracture will be investigated at the chord saddle location.

4.3 Failure Model

A selection of the failure model (limit state function) is the most important step in the probabilistic analysis. The failure criterion qualifies various material properties and various types of loadings as more or less important from the view of possible failure. The aim of the work is to show the effect of the variability in fracture toughness of the LT60 steel plate on the probability of fracture of the selected joint configurations.

Two failure criteria were selected:

1. propagation of a fatigue crack through the chord thickness; and
2. initiation of unstable fracture - the CTOD criterion.

4.3.1 Through-thickness crack propagation criterion

The fatigue crack propagation law widely used (e.g. DnV Rules, 1977) is that of Paris and Erdogan (1963). The crack growth rate, da/dN , is given by:

$$\frac{da}{dN} = C(\Delta K)^m, \quad (4-1)$$

where C, m are crack growth parameters (material constants); and ΔK is the stress intensity factor (SIF) range.

The crack growth rate da/dN denotes an extension of the crack length (depth) caused by one stress cycle. The Paris-Erdogan law and its application to crack propagation in tubular joints has been extensively studied (e.g. Vosikovsky, Bell, Burns and Mohaupt, 1985). The crack-growth parameters depend on the stress intensity ratio

$R = K_{\min}/K_{\max}$ ($K_{\min}$ and $K_{\max}$ are the minimum and maximum stress intensity factors during a stress cycle) and on the environment. DnV (1984) recommended values for the crack-growth parameters given in Table 4-2.

Table 4-2

Crack Growth Parameters

	m	C (ΔK in Nmm ^{-3/2} , a in mm)	
		Mean value	Mean + 2 st.dev.
Welds in air	3.1	$1.1 \cdot 10^{-13}$	$3.3 \cdot 10^{-13}$
Welds immersed in seawater	3.5	$3.4 \cdot 10^{-14}$	$1.6 \cdot 10^{-13}$

The Paris-Erdogan law underpredicts the crack growth if there is a significant number of cycles at high stress intensities ($K_{\max} \geq 0.7 K_C$), a situation not normally encountered in tubular joints in offshore platforms (UEG, 1985). The threshold effect of SIF-range on the crack growth rate is not considered in Eq. (4-1). A crack, growing at the "hot-spot" stress of a tubular joint during the service life of a structure, is subject to very complex, ever-changing, stress and geometry conditions. A SIF for two-dimensional problems is given by:

$$K = Y \sigma_{HS} (\pi a)^{1/2} \quad (4-2)$$

where Y is a factor depending on the geometry, crack size, type of loading, etc., σ_{HS} is the "hot-spot" tensile stress normal to the crack faces, and a is the crack depth. Dover and Dharmavasan (1982) investigated experimentally the geometry factor Y as a function of the relative crack depth a/T (T = thickness) for various joint configurations and loadings. The curves can be approximated by an expression

$$Y = A(T/a)^j \quad (4-3)$$

where A and j are constants to be determined from experimental data. Figure 4-4 shows those for axial loading and T-, Y-, and K-joints.

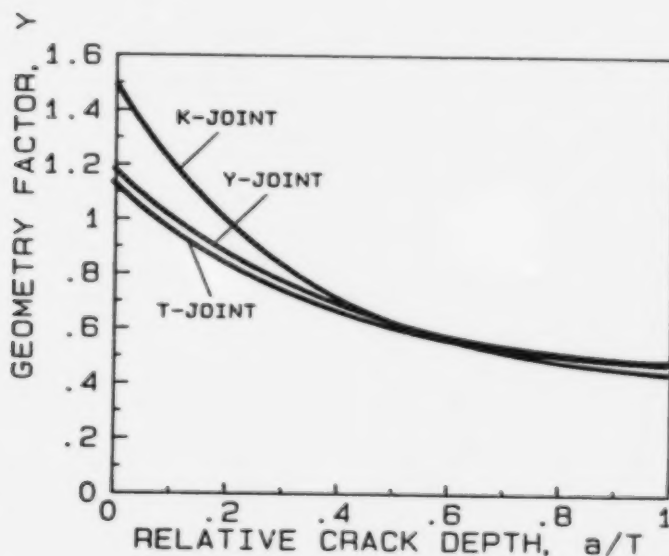


Figure 4-4. Geometry Factor for Tubular Joints - Axial Loading

For combined loading of OPB, IPB and axial, only OPB and axial loadings contribute to the propagation of the chord-saddle crack, Figure 4-5.

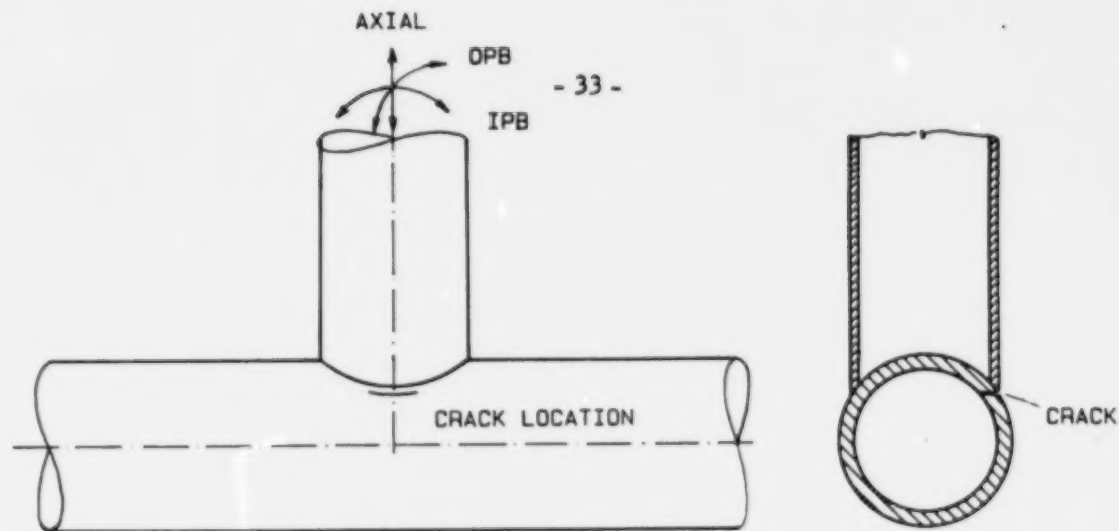


Figure 4-5. Crack Propagating at Chord-Saddle Location

The resultant stress intensity factor range is given by a superposition of the two cases (assuming that axial and OPB loadings are in phase):

$$\Delta K = Y_A(a) \Delta \sigma_{HS1} + Y_{OPB}(a) \Delta \sigma_{HS2} \quad (4-4)$$

where

$$\Delta \sigma_{HS1} = SCF_A \cdot \Delta \sigma_A \text{ and } \Delta \sigma_{HS2} = SCF_{OPB} \Delta \sigma_{OPB}.$$

Eq.(4-1) can then be rewritten in an incremental form as:

$$\Delta a = C(Y_A(a) \Delta \sigma_{HS1} + Y_{OPB} \Delta \sigma_{HS2})^m (\pi a)^{m/2} \Delta N. \quad (4-5)$$

This form is suitable for numerical integration, controlling the accuracy by limiting the maximum crack depth increment Δa_{max} . The crack depth at the time t is given by the sum:

$$a(t) = a_0 + \sum_{i=1}^{N(t)} \Delta a_i, \quad (4-6)$$

where $N(t)$ is the equivalent number of stress cycles at the time t of the life of a structure, and a_0 is the initial crack depth. Eq.(4-5) assumes the same equivalent number of stress cycles for both axial and OPB loadings.

Assuming only axial loading and $Y = \text{const.}$, Eq.(4-1) can be integrated in a closed form, giving the crack depth at time t as:

$$a(t) = (a_0^{(2-m)/2} + \frac{(2-m)}{2} \pi^{m/2} Y^m C N(t) \Delta \sigma_{HS1}^m)^{2/(2-m)}. \quad (4-7)$$

The margin of safety for the through-thickness crack failure criterion can be expressed as:

$$g = T - a(t) \quad (4-8)$$

where $a(t)$ is given by Eq.(4-6) or Eq.(4-7). Thus, the probability of failure (probability that the fatigue crack breaks through the thickness at the time t) is given by:

$$P_f = P(T - a(t) < 0) \quad (4-9)$$

4.3.2 CTOD - Fracture criterion

The margin of safety for unstable fracture, based on the crack-tip opening displacement (CTOD), can be expressed in terms of a maximum crack depth tolerated by the tubular joint, $a_{\max}$, and the effective crack depth a_{eff} as:

$$g = I a_{\max} - a_{\text{eff}}, \quad (4-10)$$

where I is the model uncertainty factor accounting for the conservatism in the CTOD-design curve. Based on 97% survival, the uncertainty factor was shown to be in the range 2 to 3 (Kamath, 1981). However, it was shown that this factor can be as high as 12 for a yield strength mismatch of 40% between the weld and the parent

steel (Lereim, 1985). It can be expected that the complex stress field and geometry of tubular joints (as compared to wide plate tests) would lead to higher model uncertainty factors, certainly outside of the commonly accepted range of 2 to 3.

The probability of failure, given by the CTOD criterion, is then

$$P_f = P(Ia_{\max} - a_{\text{eff}} < 0) \quad (4-11)$$

The maximum crack size, $a_{\max}$, corresponding to the CTOD-design curve, proposed originally by Burdekin and Dawes (1971), depends on the critical value of CTOD, δ_c , the maximum crack-opening stress, $\sigma_{\max}$, and the yield strength, σ_y , of the material in which the crack propagates (parent material or weld):

$$a_{\max} = \frac{E\delta_c}{2\pi(\sigma_{\max} - 0.25\sigma_y)} \text{ for } \sigma_{\max} > 0.5\sigma_y. \quad (4-12)$$

The maximum stress in which the crack propagates, σ , is determined by the "hot-spot" stress and the residual stresses, σ_{res} from welding, so that for axial loading:

$$\sigma_{\max} = \text{SCF}_A \sigma_A + \sigma_{\text{res}}. \quad (4-13)$$

The residual stress is usually expressed as a fraction of the yield strength

$$\sigma_{\text{res}} = A\sigma_y, \quad (4-14)$$

where the factor A is in the range 0 to 1. Values close to zero are used for the stress-relieved condition while values close to 1 for the as-welded condition.

The effective crack depth, a_{eff} , can be obtained from the crack depth $a(t)$, Eq.(4-7), applying appropriate correction factors as:

$$a_{\text{eff}} = (YM_w)^2 a(t), \quad (4-15)$$

where Y is the geometry function and M_w is a magnification factor due to the weld. This effect decays exponentially as the crack propagates through thickness (Cunha, Farias, and Lereim, 1984):

$$M_w = SCF_w - (SCF_w - 1)(1 - 0.7 \exp(-10a(t)/T)), \quad (4-16)$$

where SCF_w is the stress concentration factor due to the weld. Assuming $SCF_w = 2.5$, the expression (4-16) simplifies to:

$$M_w = 1 + 1.05 \exp(-10a(t)/T).$$

Table 4-3 shows the variation of M_w as a function of the relative crack depth a/T . As can be seen from this table, the stress magnification effect of the weld ceases to be felt by the crack tip after the crack propagates halfway through the thickness.

Table 4-3
Variation of M_w as a Function of a/T

a/T	0.001	0.01	0.05	0.1	0.5	0.75	1.0
M_w	2.04	1.95	1.64	1.39	1.01	1.00	1.00

4.3.3 Application of failure criteria

The failure criterion, Eq.(4-9), was introduced to investigate the propagation of an initial flaw through the thickness of the chord at the saddle location. Both the closed form solution, Eq.(4-7) and the numerical integration, Eqs. (4-5) and (4-6) have been used in developing corresponding LIMST subroutines. However, while using the whole probability distributions of the basic quantities, the numerical integration in LIMST subroutine considerably increases the computer time. Thus, for the through-thickness crack propagation, the crack depth given by Eq.(4-7) has been used.

Also, it should be noted that the terms in the square brackets in Eq.(4-7) have opposite signs ($m > 2$) and the exponent is negative. As a consequence, when these two terms approach the same absolute value, the crack depth becomes very large, see Figure 4-6. This creates problems when using the crack depth given by Eq.(4-7) in the CTOD criterion, Eq.(4-11). To provide an input to the CTOD criterion, Eq.(4-7) was solved deterministically for three initial crack depths $a_0 = 0.8$ mm, 1 mm and 3 mm, three values of the geometry parameter $Y = 0.6, 0.8$ and 1.12, and various values of the "hot-spot" stress, and figures similar to that of 4-6 were obtained (Appendix C, Figures C-1 to C-9). The resultant crack depths as functions of time (number of years in service), were then randomized using a normal probability distribution function with 10% coefficient of variation.

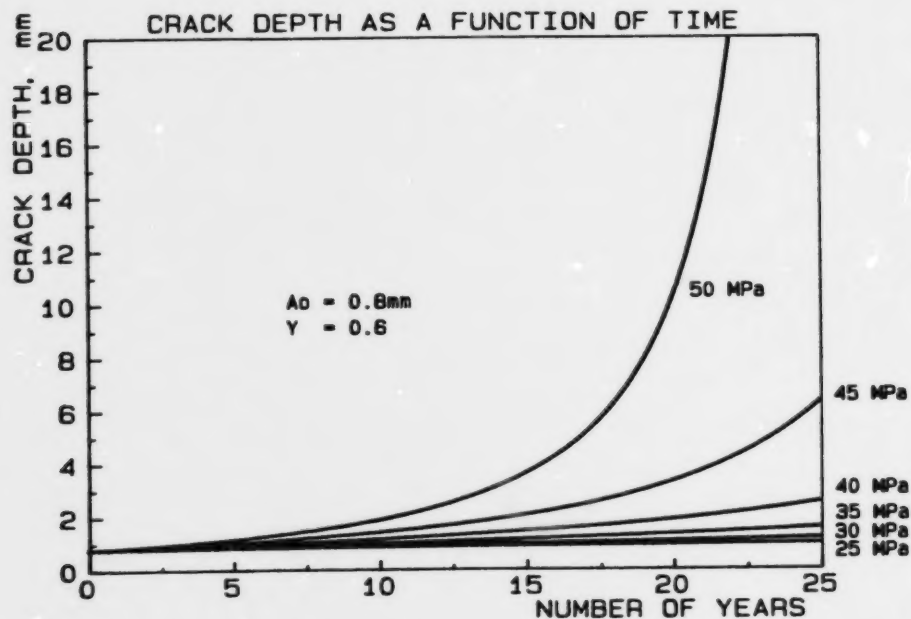


Figure 4-6. Crack Depth as a Function of Time

The choice of individual basic quantities, their statistical representation and the implications for the failure criteria will be discussed in detail in the following section.

4.4 Input Data - Basic Quantities

4.4.1 Initial flaw size

For any investigation of fatigue crack propagation, the knowledge of the initial flaw (crack) size is of paramount importance. The probability of finding a flaw of a given size (introduced in the structure during fabrication) will be related directly to the detection probability of the non-destructive evaluation (NDE) method applied (Bokalrud and Karlsen, 1982). It has been shown (Wong and Rogerson, 1983) that Weibull probability distribution can be used to define the weld defect size distribution, **Figure 4-7**. This distribution gives conservative values for large defect sizes which are most important for the determination of the probability of failure. The probability density function for Weibull distribution, $f(x)$, is given by:

$$f(x) = \beta/\eta ((x - \gamma)/\eta)^{\beta-1} \exp \{ -((x - \gamma)/\eta)^{\beta} \}$$

where β , γ and η are parameters of the distribution.

The Weibull probability distribution, with parameters $\beta = 0.8$, $\gamma = 0.1$ mm and $\eta = 1.12$ mm, was used in the through-thickness criterion. For comparison, also the normally distributed initial flaw size with the mean value of $\mu_{a_0} = 1$ mm and 10% coefficient of variation, was used.

As already mentioned in Section 4.3.3, the crack size during service was obtained from Eq. (4-7) using three different initial crack sizes, different values of the geometry parameter and variable "hot-spot" stress ranges. The crack sizes obtained were randomized assuming normal probability distribution and the coefficient of variation of 10%.

4.4.2 Geometry factor

The geometry factor, Y , was discussed in detail in Section 4.3.1. In the case of a through-thickness crack propagation, the mean value $\mu_Y = 1.12$ with coefficient of variation of 10% were used. For crack depths as functions of time for the CTOD criterion, deterministic values $Y = 0.6, 0.8$ and 1.12 were used in the analysis.

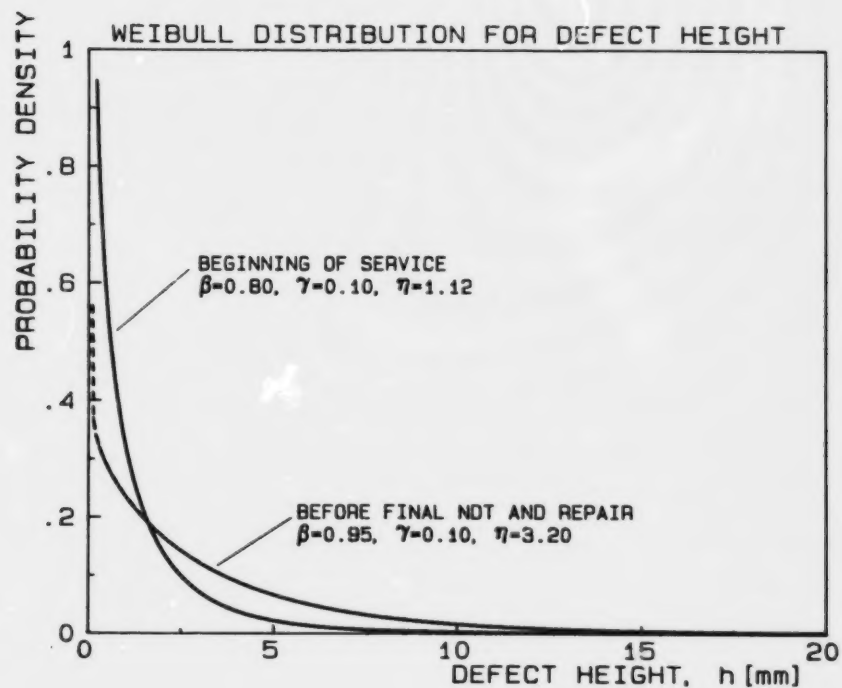


Figure 4-7. Weibull Distribution of the Defect Size

4.4.3 Crack growth parameters

The material crack growth parameter m in Eq. (4-1) is commonly taken deterministically (e.g. DnV, 1977) and all the scatter in the crack growth is assigned to the crack growth parameter C , Table 4-2. The uncertainty in the parameter C is twofold:

- uncertainty in the mean value of C for the specific material; and
- variations in C from specimen to specimen.

To account for these uncertainties, the distribution of C is assumed lognormal and $\ln C$ is expressed as (Madsen, Skjong and Kirkemo, 1985):

$$\ln C = E(\ln C) + k_1 U_1 + k_2 U_2$$

where U_1 and U_2 are independent and standardized normal variables, both with 10% coefficient of variation. $k_1 U_1$ reflects the variations in C from specimen to specimen and $k_2 U_2$ reflects the uncertainty in the mean value of C . Based on 25 test results, Tanaka et al. (1981) have estimated K , as 0.255. Based on the values for air in Table 4-2, the mean of $\ln C$ and the constant k_2 can be calculated so that

$$\ln C = -29.83 + 0.255 U_1 + 0.543 U_2$$

This expression was used in the analysis.

4.4.4 "Hot-Spot" Stress Range

The loads acting on offshore structures are mostly of a dynamic character. Generally, the following effects need to be considered (Thoft-Christensen and Baker, 1982):

- extreme wind speed;
- extreme current;

- spectral form of the extreme sea-state;
- extent of marine growth;
- hydrodynamic forces; and
- dead and live deck loads.

Having no load input in the analysis, it is of interest to determine the maximum allowable cycling loading on the joint. From the chart, **Figure 4-8** (DnV, 1984), we can determine the allowable stress amplitude during the 20-year service life as $\sigma_{amp} = 107$ MPa (fatigue class T and $h = 1.1$, see Eq. (4-17) for definition of h). The maximum allowable stress range is then $\Delta\sigma = 2 \sigma_{amp} = 214$ MPa. This value should be thickness adjusted (class T results were obtained for

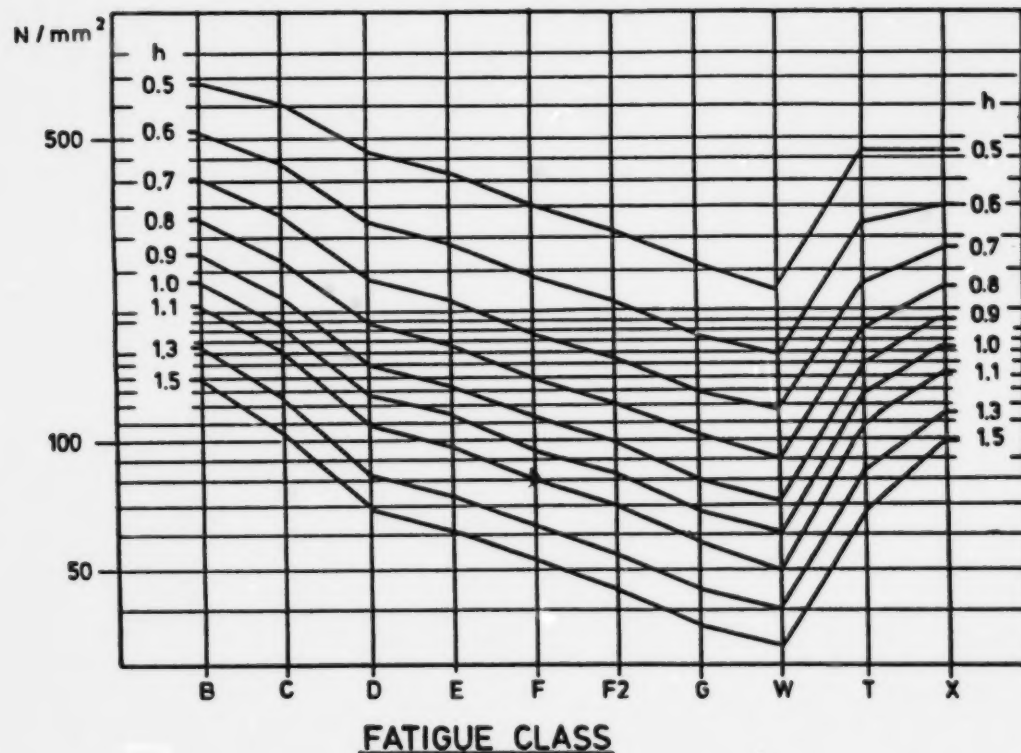


Figure 4-8. Allowable Extreme Stress Amplitude (20 years)

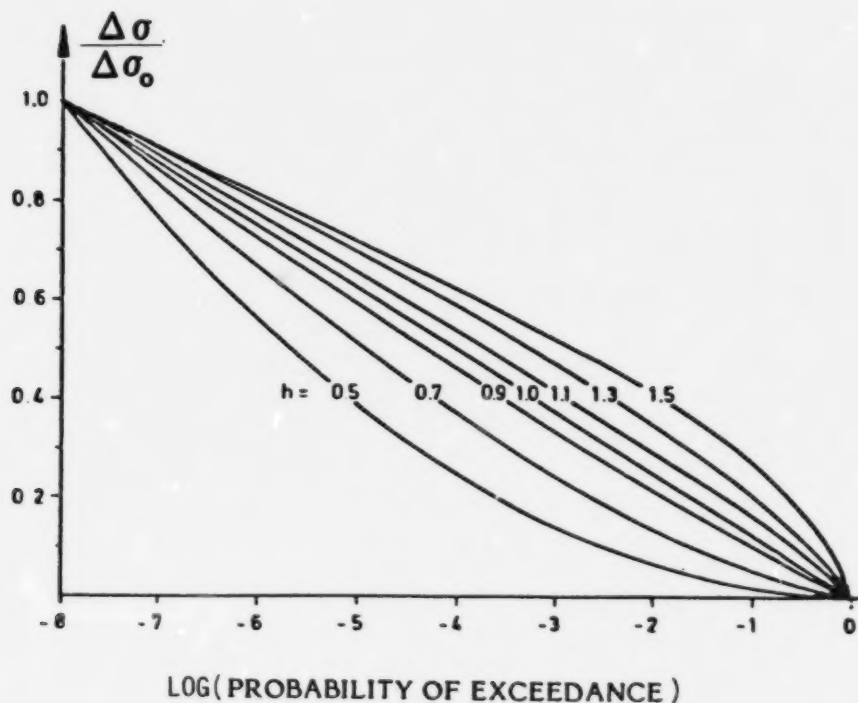


Figure 4-9. Long-Term Distribution of Stress Range

$T = 32 \text{ mm}$) so that $\Delta\sigma_0 = 214(32/T)^{1/4} = 302.6 \text{ MPa}$ for $T = 16 \text{ mm}$. This is the maximum amplitude exceeded once in N_0 stress variations in the stress range distribution

$$\Delta\sigma = \Delta\sigma_0 (1 - \log N / \log N_0)^{1/h} \quad (4-17)$$

where N is a number of stress variations in which σ is exceeded, h is a Weibull parameter, see Figure 4-9, and N_0 is the total number of cycles endured by the structure during the 20 years of service life.

Assuming an average stress cycle period of 10 s, the total number of stress cycles is:

$$N_o = \frac{365 \times 24 \times 60 \times 60}{10} \times 20 = 6.3072 \times 10^7 \text{ cycles.}$$

The equivalent effective stress range representative of the long term distribution is given by (DnV, 1984) as

$$\Delta\sigma_{eq} = \frac{\Delta\sigma_o}{(\ln N_o)^{1/n}} \left(\Gamma \left(\frac{m}{h} + 1 \right) \right)^{1/m} \quad (4.18)$$

where m is the crack growth parameter ($m = 3.1$), and $h = 1.1$, as defined earlier.

Evaluating Eq. (4.18), the equivalent stress range is $\Delta\sigma_{eq} = 36.4$ MPa. The range of equivalent "hot-spot" stress range values, used in the analysis, was from 10 to 50 MPa with 10% coefficient of variation.

4.4.5 Fracture toughness

The steel data used in the analysis was that of ALGOMA LT60. Fracture toughness of this steel was extensively tested both by ALGOMA Research Department (Ray, Ackert and McLean, 1984) and by CANMET (Faucher and Tyson, 1985). In the latter work, Faucher and Tyson used the J_C (J-integral toughness) measurements to correlate the K_C fracture toughness to the yield strength and consequently the temperature. To obtain the yield strength as a function of temperature, the yield strength data was correlated with temperature using the IMSL (1982) second-order correlation routine, Figure 4-10.

This functional relation was then used in the formula of Faucher and Tyson (1985).

$$\begin{aligned} \ln K_C = & 41.34 + 0.248 \ln(50/T) - 5.82 \ln \sigma_y \\ & + 0.248 \ln \ln (1/(1-\phi)) \end{aligned} \quad (4-19)$$

where T is thickness in mm, σ_y is yield strength in MPa, and K_C is fracture toughness in $\text{MPa m}^{1/2}$. ϕ is the cumulative probability of failure of randomly chosen the J_C -specimens so that all the statistical variability associated with the J_C -tests is expressed through this probabilistic parameter. For example, the CTOD design curve is based on a 97% probability that the allowable crack size does not exceed the critical crack size (see Section 4.3.2). This would correspond to $\phi = 0.03$ for the J_C -tests. The value of ϕ , however, is not identical with the probability that the allowable crack size exceeds the critical crack size in a real structure. For this reason a model uncertainty factor I was introduced in Eq. (4-11). The above formula was derived based on the statistical model of Tyson and Marandet (1985).

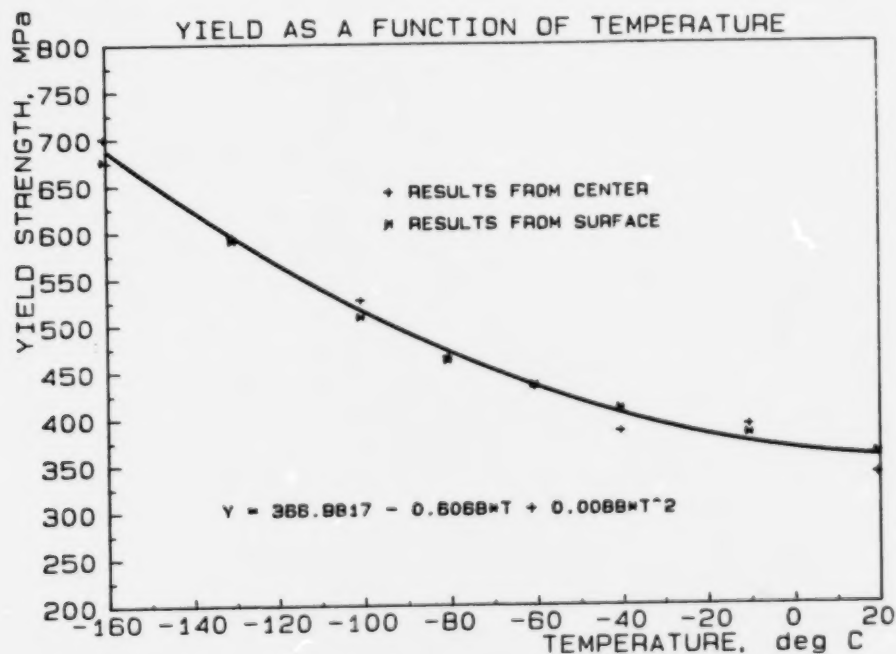


Figure 4-10. Yield Strength as a Function of Temperature

Using the relation between the CTOD fracture toughness δ_C and the K_C fracture toughness (e.g. UEG, 1985),

$$\delta_C = \frac{K_C^2 (1 - \nu^2)}{2E \sigma_Y} \quad (4-20)$$

δ_C is plotted as a function of temperature, thickness and statistical parameter ϕ , Figures 4-11, 4-12, and 4-13.

The fracture toughness calculated from Eq. (4-19) is that of LT60 steel plate and is relevant to the condition of unstable fracture when the crack propagates through the parent material.

The CTOD fracture toughness for K-PREP welds (coarse grain HAZ) was tested for different thicknesses and temperatures (Ray, Ackert and McLean, 1984). With respect to the limited data available, results obtained for 25 mm thickness and two temperatures, -35° and -60°C have been used in the present work, Table 4-4. The values show

Table 4.4
 δ_C Fracture Toughness for K-PREP Welds for 25 mm Thickness

Temperature ($^\circ\text{C}$)	Number of Tests	Mean (mm)	Std. Dev. (mm)	Coeff. of Variation (%)
-35	3	0.383	0.278	72.6
-60	7	0.103	0.051	48.9

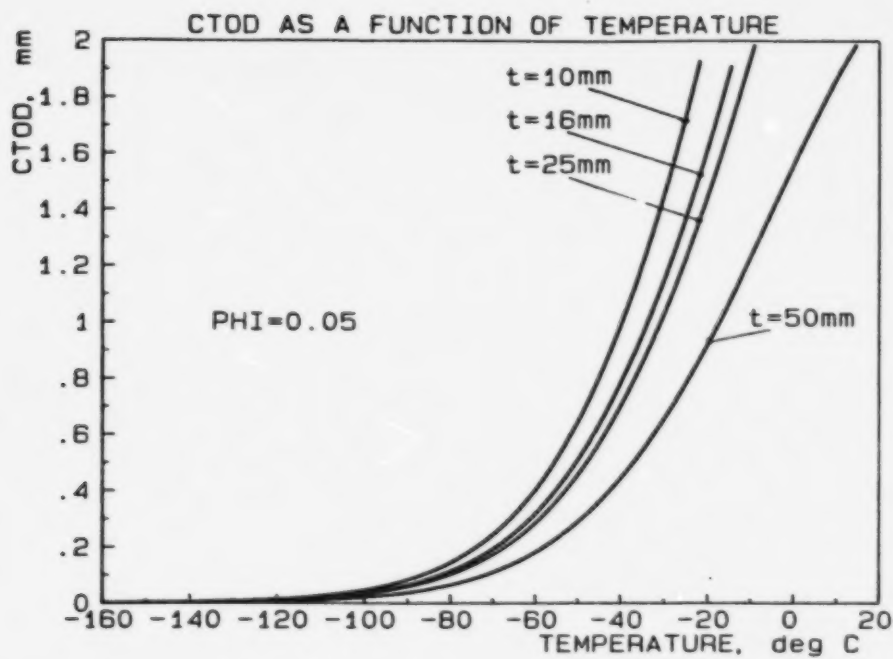


Figure 4-11. δ_C as a Function of Temperature, $\Phi = 0.05$

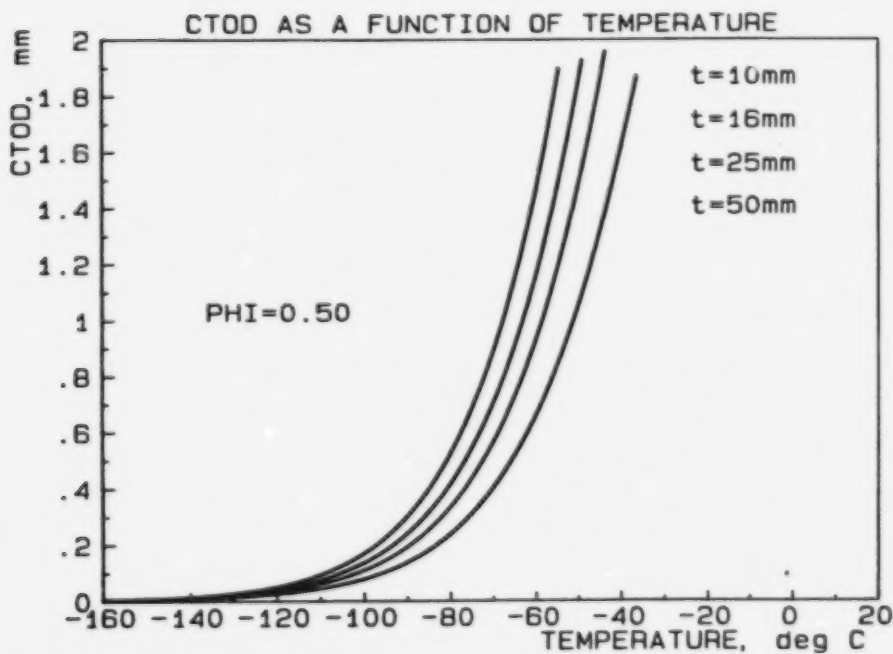


Figure 4-12. δ_C as a Function of Temperature, Φ

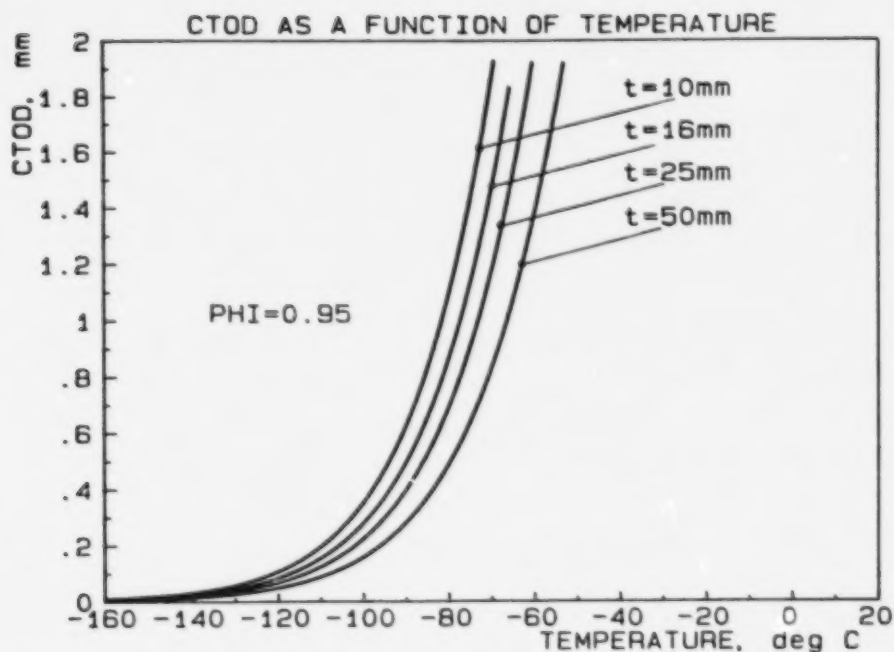


Figure 4-13. δ_C as a Function of Temperature, $\phi = 0.95$

a considerable scatter. One test value, $\delta_m = 0.716$ (-60°C), was disregarded as this value corresponded to the attainment of the maximum force plateau. This value would change the mean and standard deviation so that a coefficient of variation of more than 100% would result at -60°C .

4.4.6 Residual stresses

As already discussed in Section 4.3.2, Eq. (4-14), the residual stress is taken as a fraction of the yield strength. For this project, the value of $A = 0.7$ with 10% coefficient of variation was used. Residual stress equal to 70% of the yield strength is often used as representative of the as-welded condition (Madsen, Skjong, and Kirkemo, 1985).

4.4.7 Nominal Stress in the Brace

For the criterion of unstable fracture, Eq. (4-11), the maximum (peak) nominal stress in the brace was taken according to API Recommended Practice (API RP2A, 1984) as 138 MPa, with 10% coefficient of variation. This value was then magnified, using an appropriate SCF, and superimposed on the residual stress.

5. DISCUSSION OF THE RESULTS

5.1 General Comments

Using various combinations of the input data, numerous results were generated. To make the results more readable, they were post-processed and they are presented in a self-explanatory graphical form. Only the most representative figures are presented in the main text, while the rest was placed conveniently in APPENDIX C. **Figures C-10 to C-54** show the probability of failure as a function of time for the T-joint with $\phi=0.05$ and **Figures C-55 to C-66** for the T-joint and $\phi=0.50$. **Figures C-67 to C-78** show the probability of failure as a function of time for the Y-joint ($\phi=0.05$) and **Figures C-79 to C-90**, similar curves for the K-joint. Finally, **Figures C-91 to C-92** show the probability of failure as a function of temperature, for the T-joint with $\phi=0.05$. The values of other identifying parameters are given on top of the figures. A typical result of the PROBAN analysis, together with explanatory notes, is presented in APPENDIX B.

5.2 Through-Thickness Crack Propagation

The through-thickness crack propagation criterion, Eq.(4-9), was included in the analysis to show how the probability of failure (probability of a crack breaking through the thickness) increases with time in service. The Weibull distribution of the initial flaw size at the beginning of service, **Figure 4-7**, allows for the presence of large initial cracks and the probabilities of a crack breaking through thickness are relatively high, even for small stress ranges, **Figure 5-1**. A truncated Weibull distribution would give more realistic results. For example, Fredheim (1983) suggests a truncation level of $0.9 \times$ thickness.

The "hot-spot" stress range of 5 MPa seems to be a threshold under which there is no increase of P_f during the service life of the structure.

To demonstrate the effect of the initial flaw size, a case with normal distribution of the initial flaw size, with the mean value $A_0 = 1$ mm and 10%

coefficient of variation, was analyzed, Figure 5-2. It should be noticed that the probabilities of failure in the latter case approach those obtained for the Weibull distribution only at high "hot-spot" stress ranges.

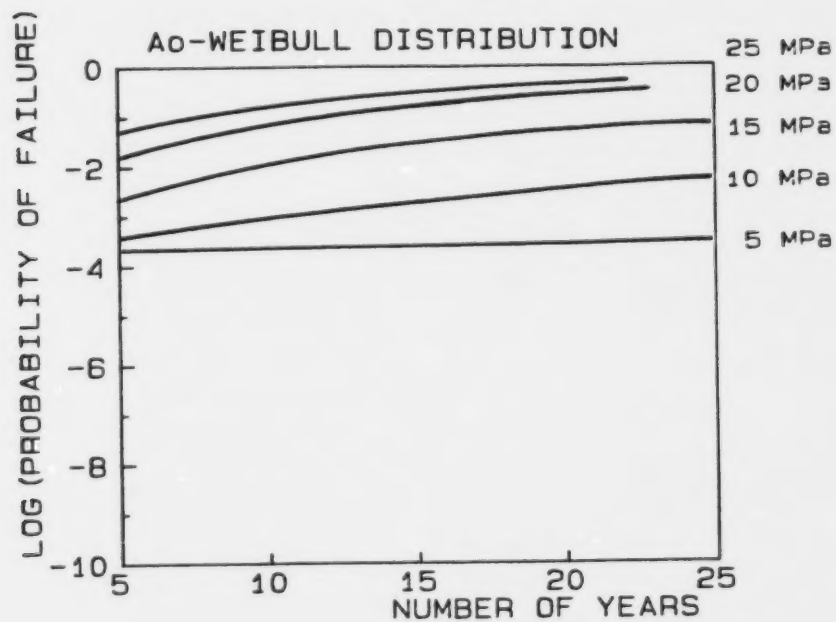


Figure 5-1. Through-Thickness Crack Propagation
Weibull Distribution, $\beta = 0.8$, $\gamma = 0.1$, $\eta = 1.112$

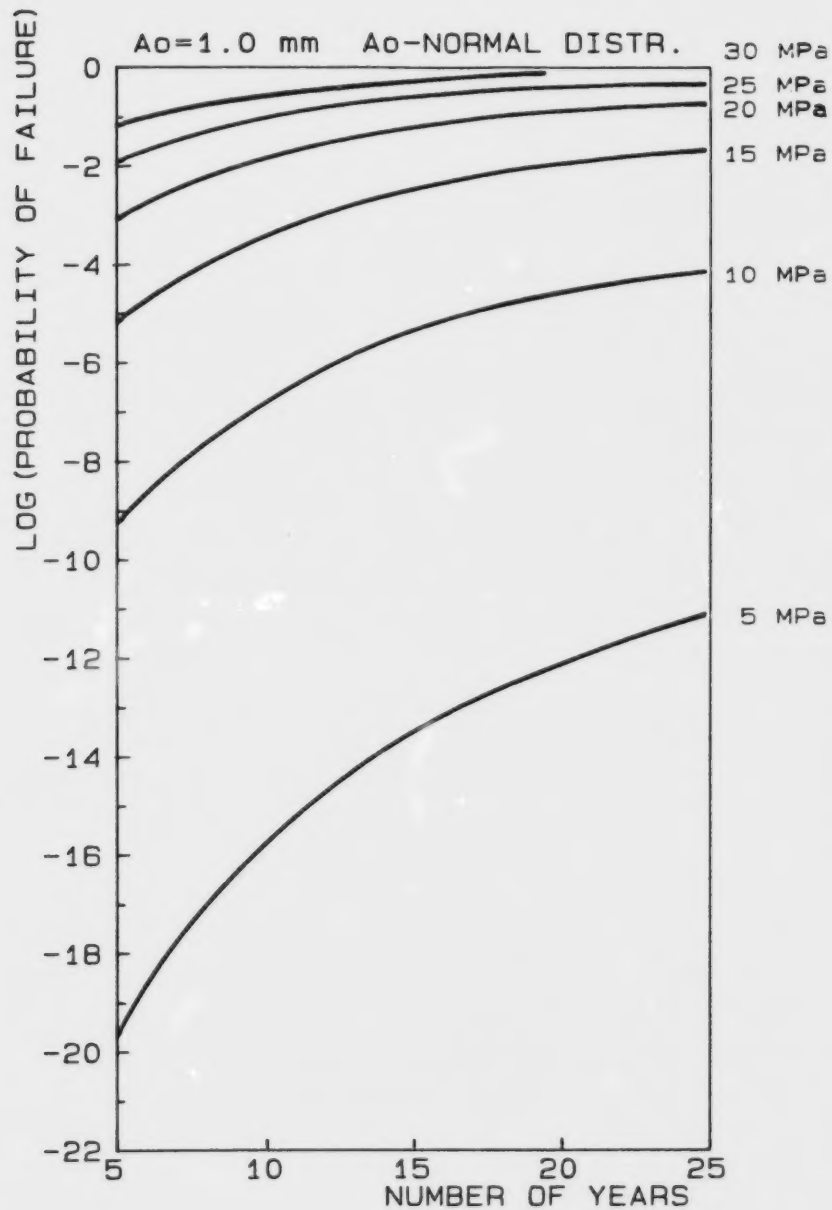


Figure 5-2. Through-Thickness Crack Propagation
Normal Distribution, $A_0 = 1$ mm, $V = 10\%$

5.3 Unstable Fracture

This analysis was based on the CTOD failure criterion, Eq. (4-11). Fracture toughness (CTOD) for the analysis was obtained either from Eq. (4-19) (parent steel), which allowed an investigation of the probability of failure as a continuous function of temperature, or the CTOD data were input directly (weld).

The S-curves for parent steel, Figure 5-3, indicate a very high sensitivity of the probability of failure to temperature.

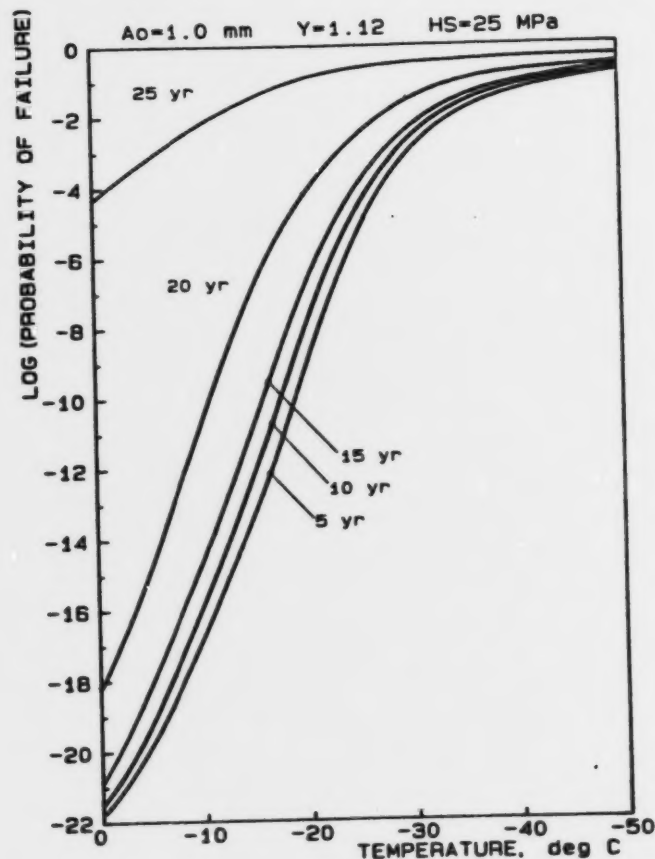


Figure 5-3: Probability of Failure as a Function of Temperature (S-Curves). T-Joint

The closeness of the curves for 5, 10 and 15 years time interval should be observed. This fact reflects the initial slow crack propagation. The S-curves show a steep transition region and an upper level characterized by high probability levels. Figure 5-4 shows the

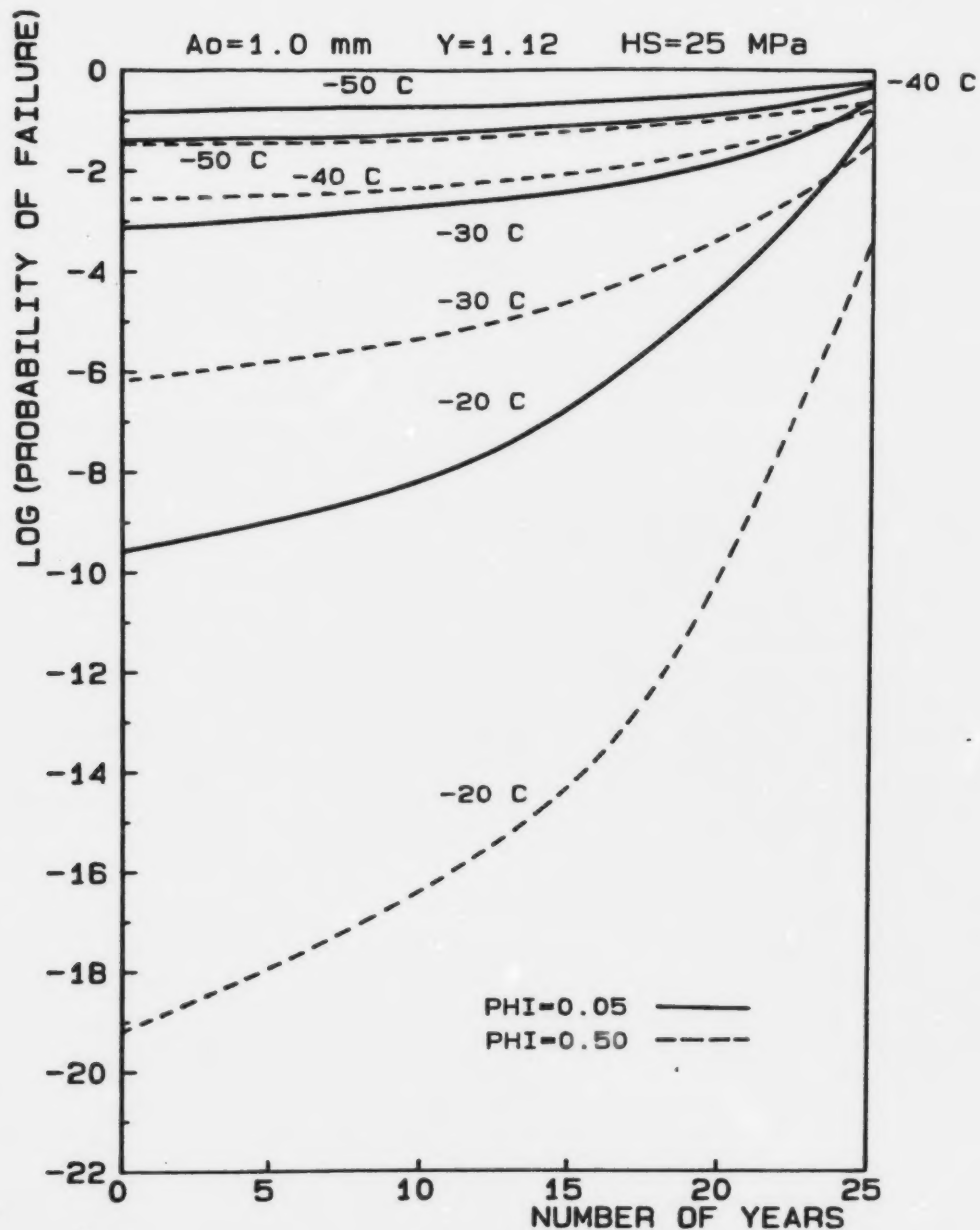


Figure 5-4: Probability of Failure as a Function of Time
T-Joint. Parent Steel.

probability of failure as a function of time, and compares the curves for two values of ϕ ($\phi = 0.05$ and 0.5 respectively). The difference between these two cases in terms of probabilities of failures decreases when approaching low temperatures (-50°C). Also, the curves flatten out at low temperatures as a consequence of decreased sensitivity to time. The decreasing of fracture toughness (lowering of temperature) progressively increases the probability of unstable fracture.

As discussed in Section 4.4.5, the CTOD fracture toughness of weld was measured at two temperatures, -35°C and -60°C , see Table 4-4. The probabilities of failure for the weld are compared to those for parent steel in Figures 5-5 ($\phi = 0.05$) and 5-6 ($\phi = 0.5$). It can be seen that at low temperatures, the probability of failure is very high and rather insensitive to time. The probability of failure is governed at low temperatures by fracture toughness rather than crack propagation (fatigue).

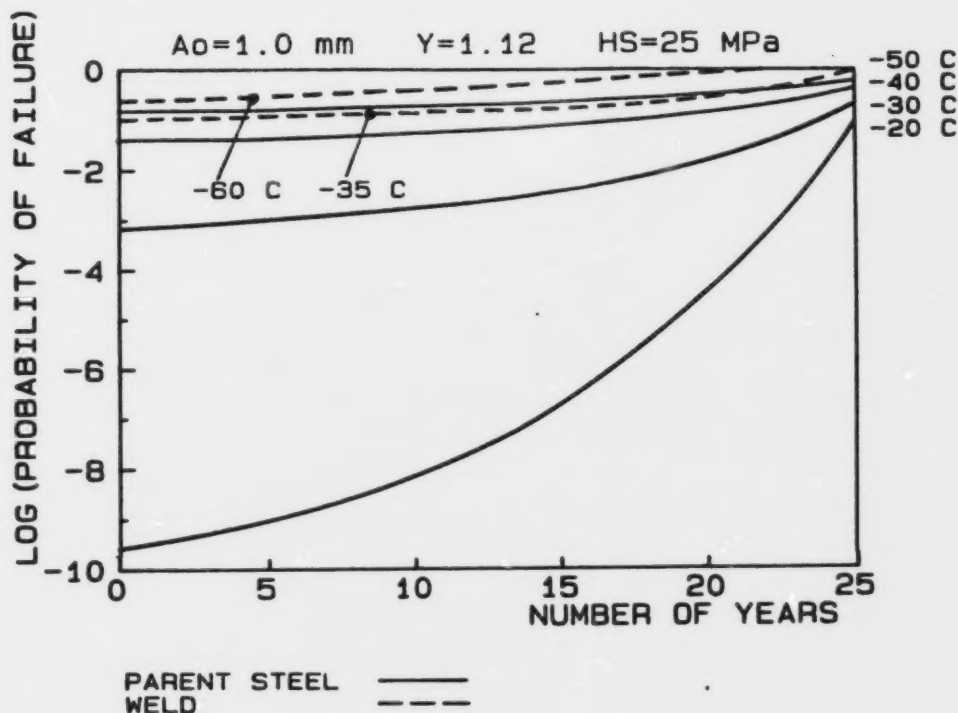


Figure 5-5: Probability of Failure as a Function of Time.
T-Joint, $\phi = 0.05$

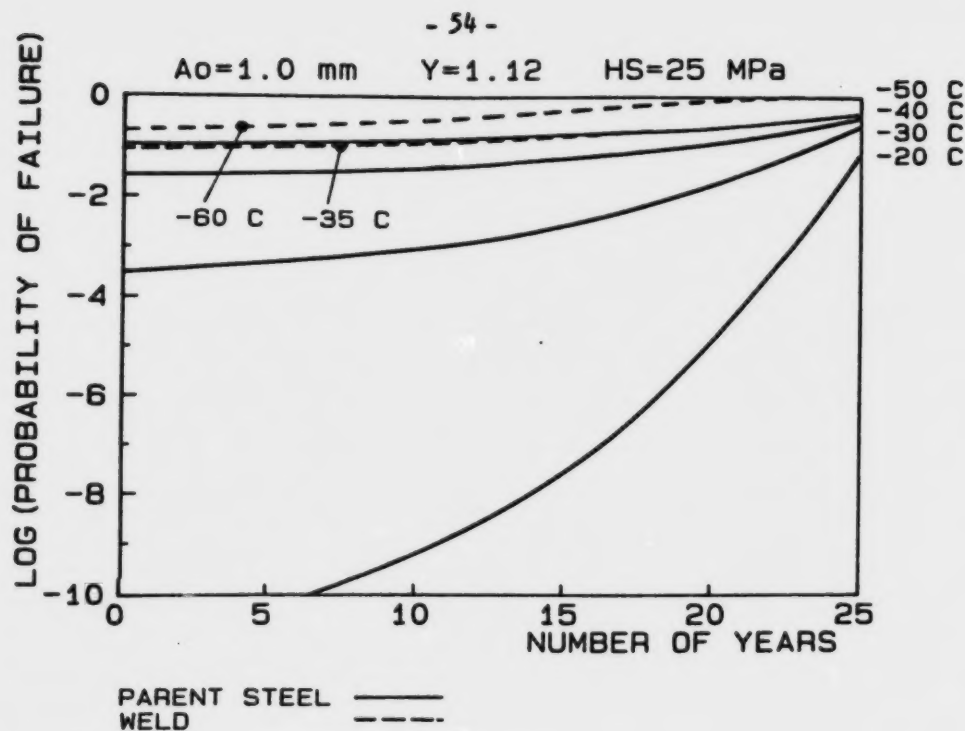


Figure 5-6: Probability of Failure as a Function of Time.
T-Joint, $\phi = 0.5$

Figure 5-7 compares probabilities of failure of the T-, Y- and K-joints at various temperatures for parent steel ($\phi = 0.05$). The different joint configurations are characterized by different SCFs, see Table 4-1. The differences between the curves in Figure 5-7 are pronounced at the beginning of the life of the joints (being separated by approximately one order of magnitude), but these differences disappear as the probabilities of failure approach the locus ($P_f = 1$) at the end of the service life.

To be in the position of developing fracture control plans for structures, it is important to have information on the relative importance of various basic quantities for the given mode of failure and how this sensitivity changes during the time of service. For example, Figure 5-8 (the notation of the basic quantities corresponds to that of Chapter 4) shows relative independence of all sensitivity factors α_i (see Eq.(2-7)) on time (at -20°C). This dramatically changes at higher temperature (0°C) as shown in Figure 5-9.

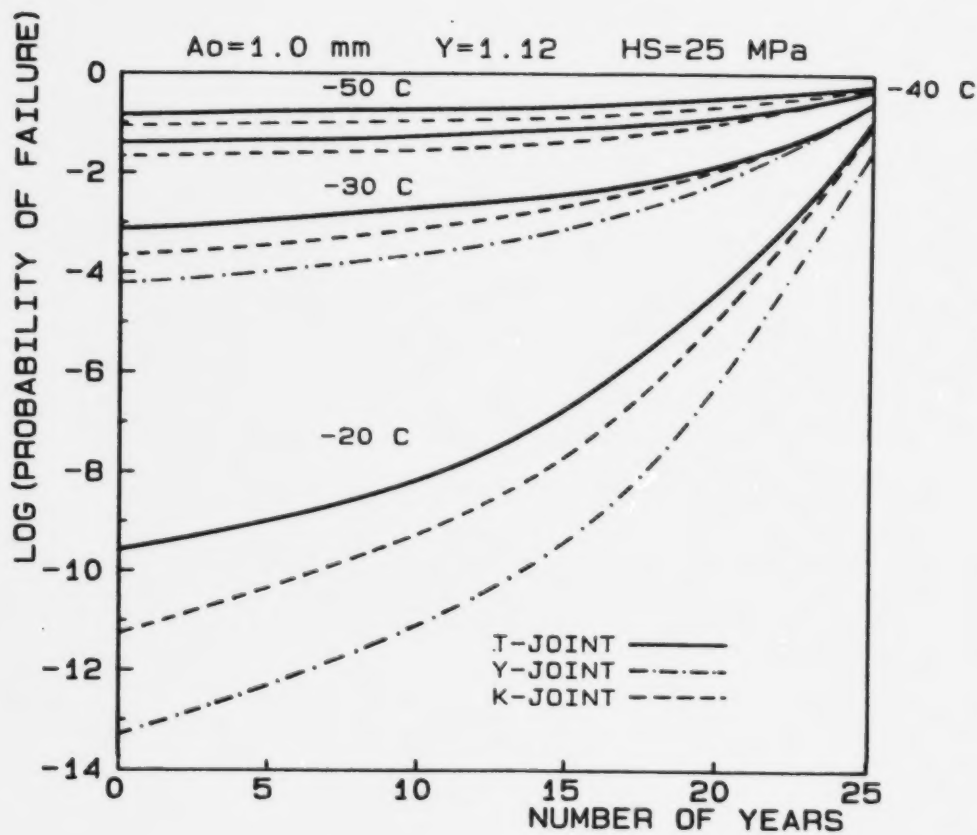


Figure 5-7: Probability of Failure as a Function of Time
T-, Y- and K-Joints, $\phi = 0.05$

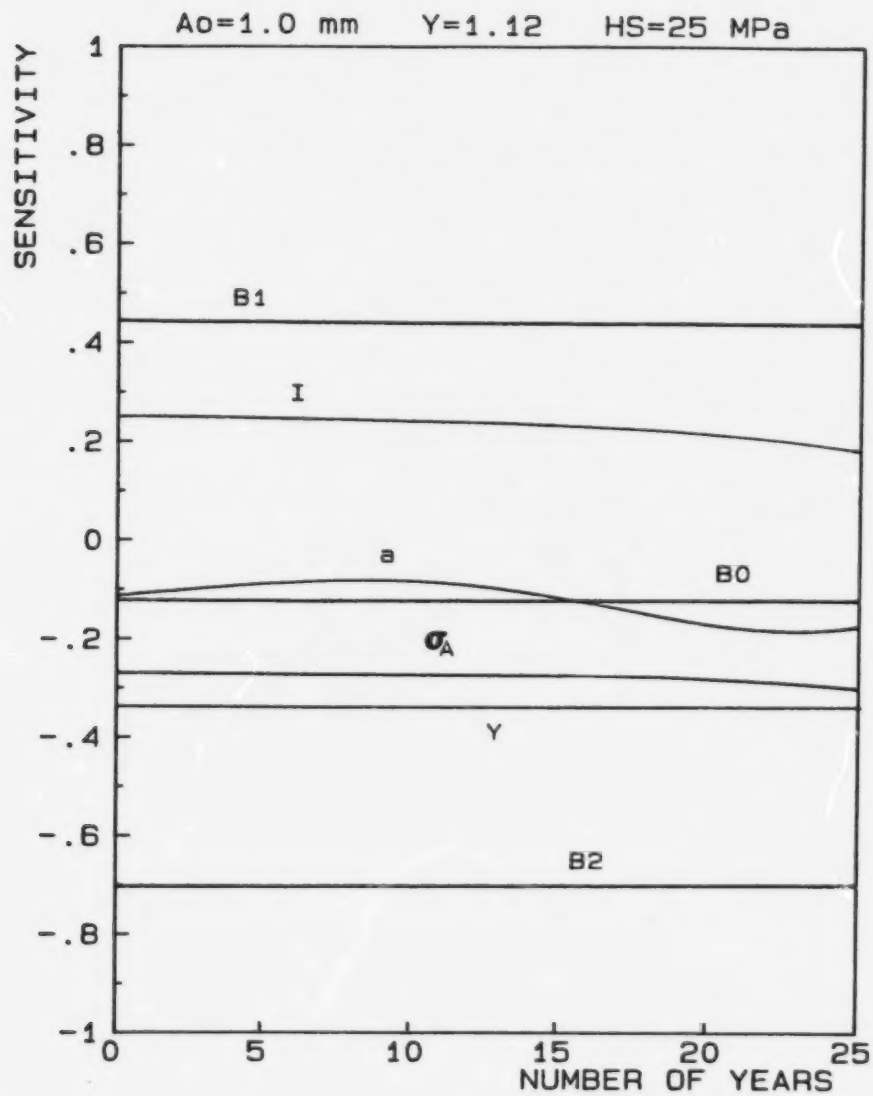


Figure 5-8: Sensitivity Factors, T-Joint, Parent Steel
Temperature = -20°C

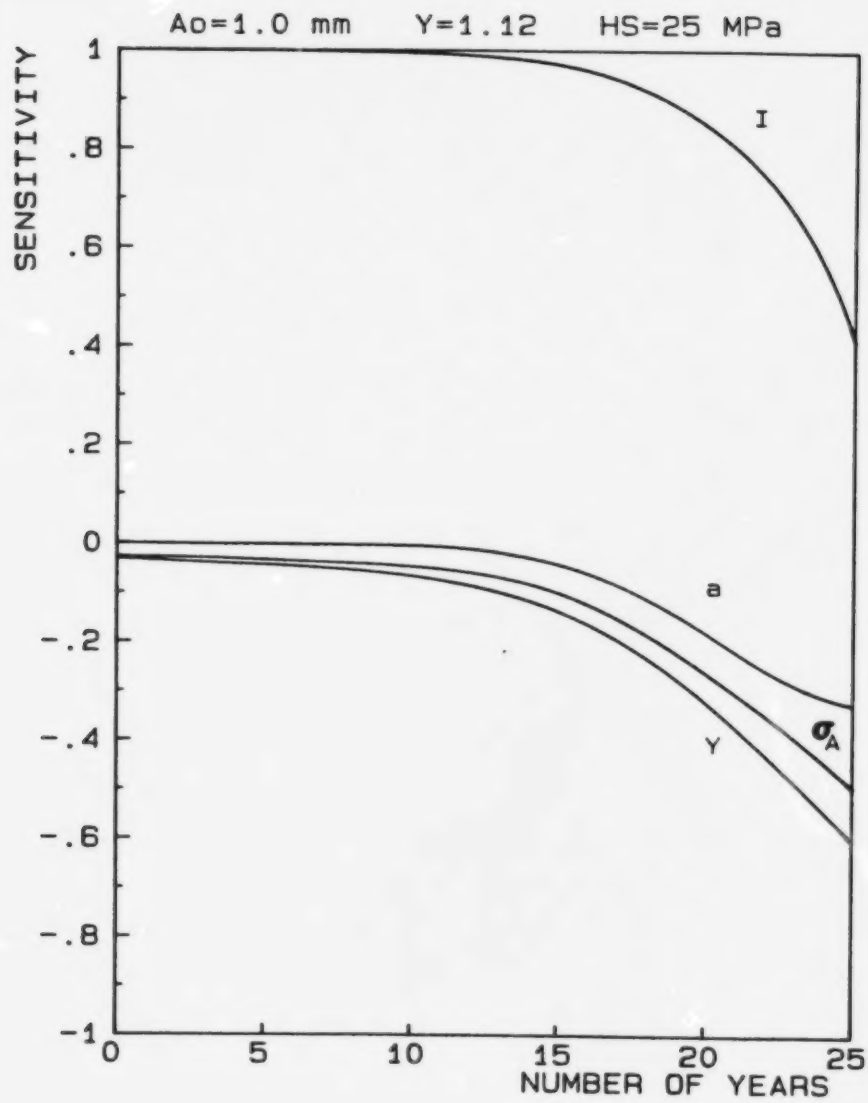


Figure 5-9: Sensitivity Factors, T-Joint,
Parent Steel, Temp. = 0°C

While at the beginning of the life of the joints, unstable fracture (parent steel) is sensitive practically only to the model uncertainty factor I, the other basic quantities take their share of "responsibility" for unstable fracture later on during the service. Temperature was input deterministically (Faucher-Tyson formula) and consequently, there is no sensitivity factor representing fracture toughness shown in these figures.

Figures 5-10 and 5-11 were obtained for crack propagation in weld at -35°C and -60°C

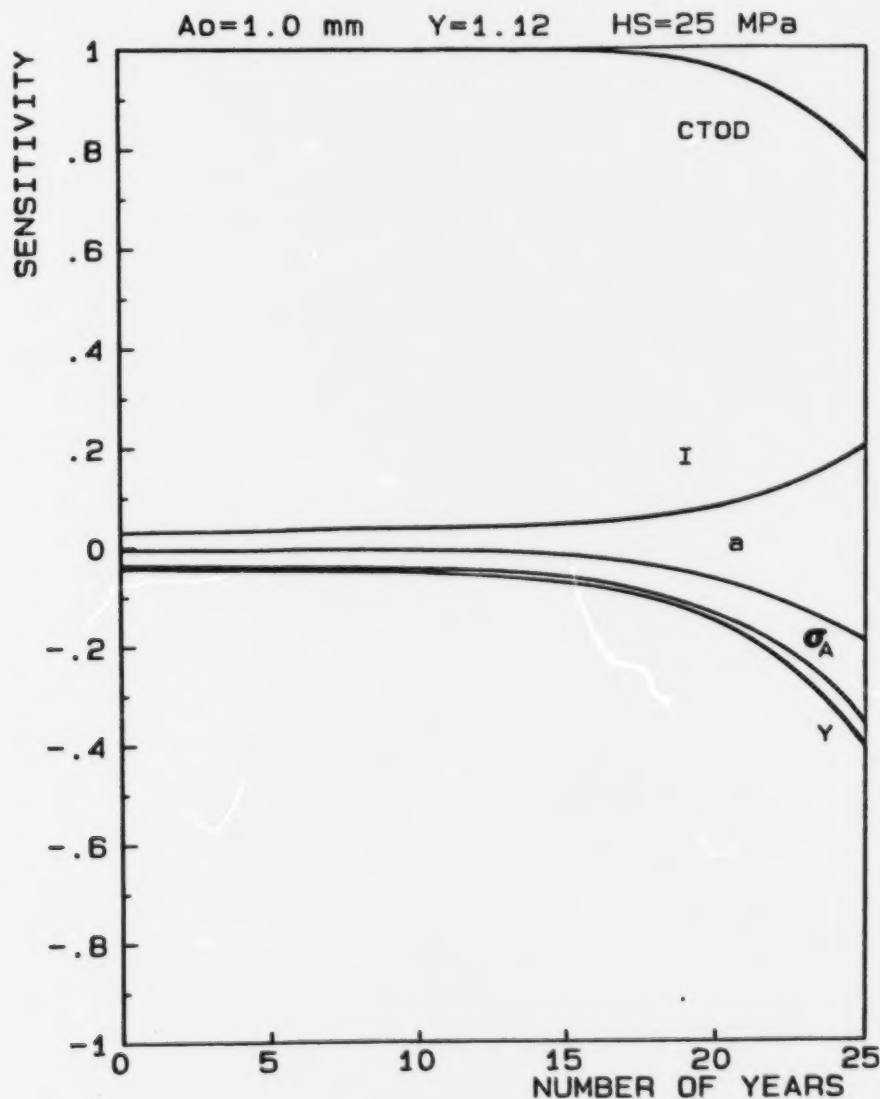


Figure 5-10: Sensitivity Factors, T-Joint, Weld, Temp = -35°C

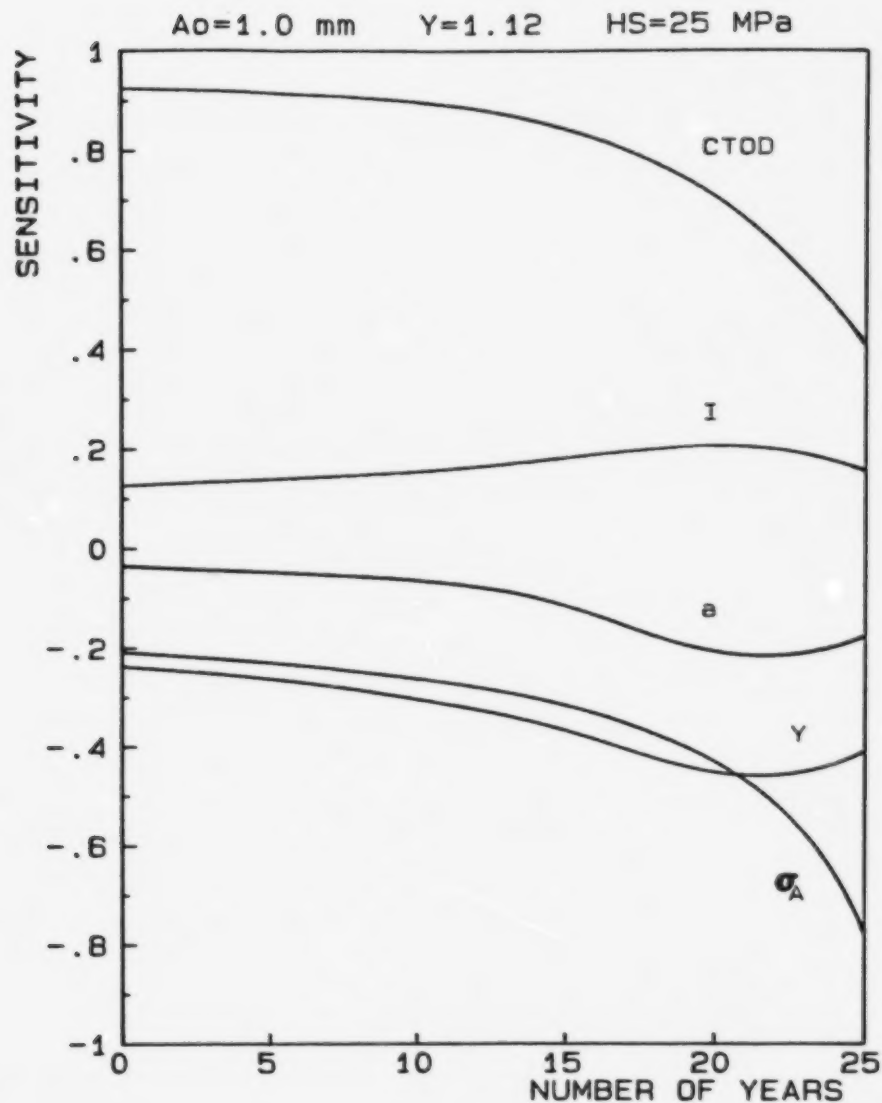


Figure 5-11: Sensitivity Factors, T-Joint, Weld, Temp. = -60°C

respectively. Here, the CTOD values were used directly in the unstable failure criterion, Eq. (4-11), and the failure is hypersensitive to fracture toughness almost during the entire life of the structure. Some increase of the sensitivity to the crack size, a , axial nominal stress σ_A and the geometry factor Y can be observed towards the end of the lifetime of the structure.

Figure 5-12 compares sensitivity factors for $I = 2.50$ and $\phi = 0.05$ at two different temperatures. The drop of the relative importance of I with decreasing temperature is highly pronounced, compare also Figures 5-8 and 5-9.

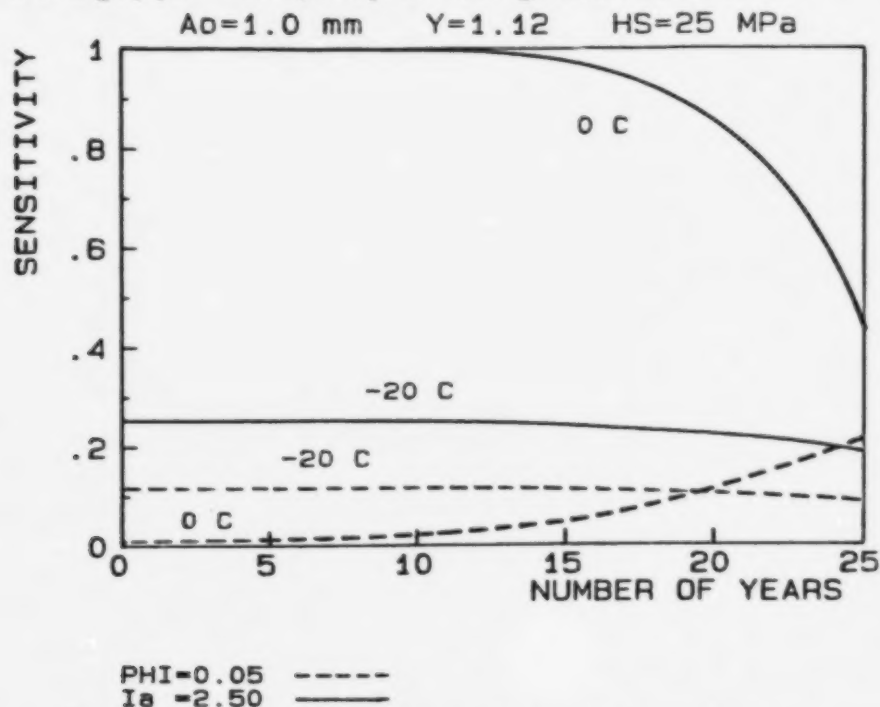


Figure 5-12: Sensitivity Factors. T-Joint, Parent Steel,
Comparison for Different Temperatures

The relative contribution of various basic quantities to failure can be shown very vividly in the bar-charts, Figures 5-13 and 5-14. Here, the squared sensitivity factors are shown so that the relative influence can be expressed on the scale 0 to 1 (see Eq. (2-8)). The charts refer to the end of service life (25 years).

Figure 5-13 shows the relative influence of basic quantities for parent steel at -20°C . Noticeable is the little importance of the residual stress factor A (quantity no. 6), model uncertainty factor I (quantity no. 5) and the initial crack size (quantity no. 4). The brace stress (quantity no. 1) has the most pronounced effect on the probability of failure in this particular case.

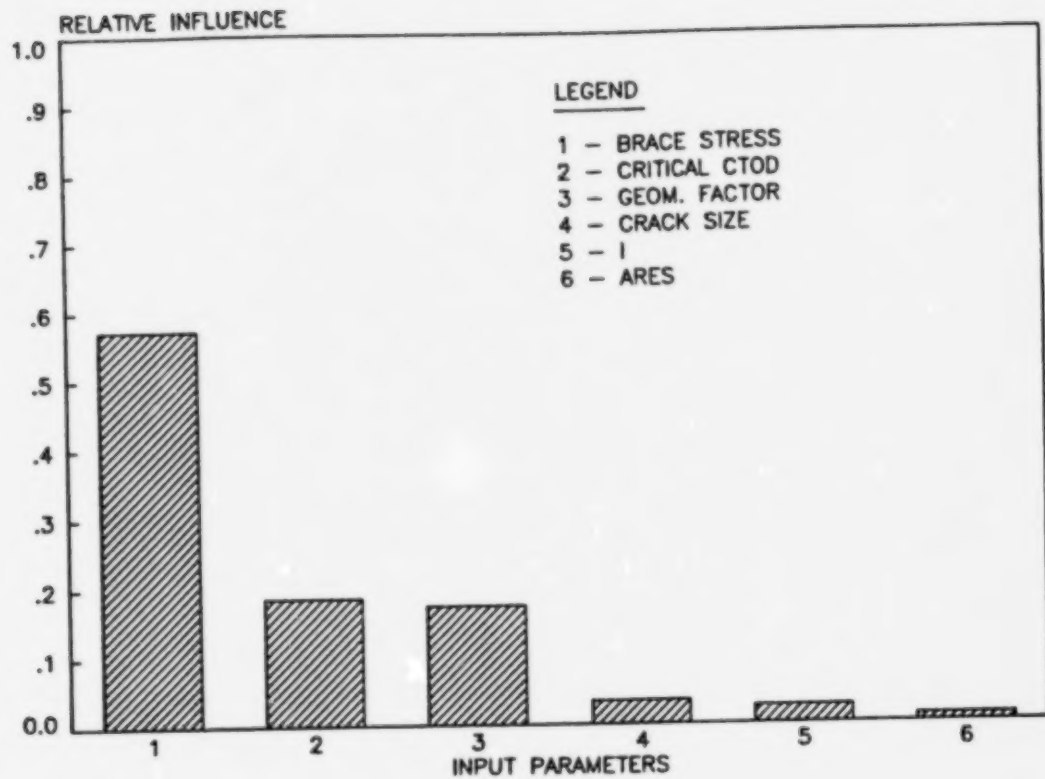


Figure 5-13: Relative Influence of Basic Quantities.
T-Joint, Parent Steel, Temp. = -20°C

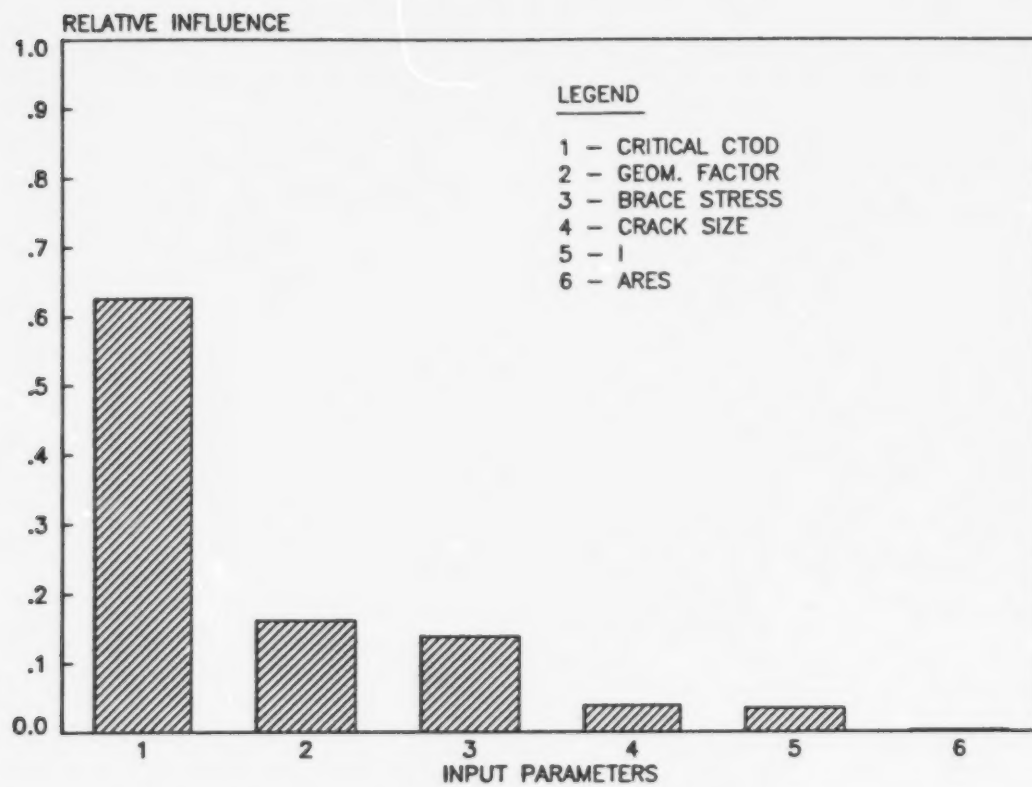


Figure 5-14: Relative Influence of Basic Quantities.
T-Joint, Weld, Temp. = -35°C

For the weld at -35°C , **Figure 5-14**, the critical CTOD (fracture toughness) is the parameter which affects the probability of failure most.

6. CONCLUSIONS

Methods of structural reliability are becoming a powerful tool in the decision process involving design, fabrication and service of structures. The applicability of these methods is particularly important when dealing with high risk environment where previous structural experience is scarce. This is true of the Canadian Arctic and subarctic regions in the context of production and transportation of hydrocarbons.

The applicability of Level 2 first-order reliability method (FORM) has been demonstrated on selected configurations of T-, Y- and K-tubular joints. The sensitivity analysis, showing the relative effect of basic variables during the lifetime of the structure, is most useful in the development of fracture control plans for structures.

While there was no direct stress (loading) input in the analysis, the work aimed at investigating the effect of the scatter in fracture toughness on the probability of unstable fracture for selected tubular joint configurations. This effect (sensitivity) was compared with that of other basic quantities and graphically documented and discussed.

At present, material requirements (toughness) for both fixed offshore structures and ships navigating in Arctic waters, is much discussed. There are no readily available steels which would meet (economically) the stringent toughness requirements (Charpy V-notch) obtained by extrapolations from moderate service temperatures. There is a justified opinion that the material requirements thus obtained are too conservative. The methods outlined in this report provide a basis for special considerations based on fracture control plans. Such approaches would lead to increased applicability of standard and high strength structural (hull) steels in low service temperatures. The stress analysis and the experience with fabrication practices provide the necessary input for such fracture control plans.

It was emphasized several times in the report how important it is to correlate calculated probabilities of failure with actual service experience. It is recommended, as a follow-up of this study, to gather information on structural failures incurred in cold waters (this includes fatigue and/or unstable fractures detected and repaired), and perform a retrospective modelling of such failures using Level 2 reliability methods. Such work would help update the calculation methods for probabilities of failure and provide a basis for structural design and for realistic application of structural steels in cold climates.

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APPENDIX A

CASES FROM THE NORTH SEA

CASE 1

OIL RIG QUALITY MOORING CHAINS

The high failure rate (up to 10%) of the Oil Rig Quality (ORQ) mooring chains used offshore in the North Sea, precipitated a comprehensive research project (Lereim, 1985). An important part of this project was the overall reliability analysis of the mooring systems. The results of this project, concerning the quality control during the entire fabrication of the chains, are reflected in the changes introduced in the DnV Classification Note on Offshore Mooring Chains (DnV, 1985).

The program of the study included the following:

- fracture strength and toughness testing;
- analysis of fatigue crack propagation;
- stress analysis and line tension analysis;
- reliability of the applied NDE method;
- size distribution of defects; and
- overall reliability analysis.

The load spectra were estimated for extreme loads as well as for the cyclic line tension of various water depths in the North Sea. Three basic loading cases were analyzed:

1. link in tension;
2. tension/bending of vertical link in the fairlead; and
3. tension/bending of horizontal link in the fairlead.

A typical design of a chain link is shown in **Figure A-1**.

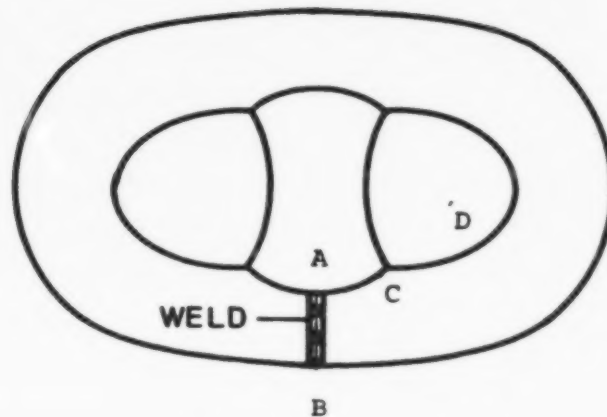


Figure A-1. Local Regions Most Likely to Contain Defects

Reliability models for single chain links, chains (cables), as well as for mooring systems have been developed. The failure criteria applied included:

1. unstable fracture;
2. fatigue fracture; and
3. plastic collapse.

Unstable Fracture

Four different fracture criteria were used:

- Linear-elastic fracture condition:

$$M = g(\underline{Z}) = K_{IC} - K;$$

- Critical nominal strain at fracture:

$$M = g(\underline{Z}) = \epsilon_C - \epsilon; \text{ and}$$

- CTOD fracture criterion:

$$M = g(\underline{Z}) = I a_{\max} - a_{\text{eff}}.$$

Fatigue Fracture

The probabilistic fatigue fracture analysis was based on the Palmgren-Miner criterion:

$$M = g(\underline{Z}) = 1 - \sum \frac{n_i}{N_i}$$

where n_i = number of cycles at a certain load level; and
 N_i = number of cycles to failure at the same level.

Ultimate Collapse Load

Failure is defined when the applied load L exceeds the critical ultimate collapse load L_u of the remaining ligament ahead of the defect:

$$M = g(\underline{Z}) = L_u - L$$

where

$$L_u = L_u^o \left(1 - \frac{A}{A_o}\right)^n$$

and

L_u^o = general yield load when there is no defect;
 A = area of defect;
 A_o = cross sectional area (no defect); and
 n = 1 for pure tension and
 2 for pure bending

Modelling of Uncertainties

The sensitivity analysis included variations in the following basic quantities:

- fracture toughness, K_{IC} ;
- ultimate collapse load, L_U ;
- fracture toughness, CTOD;
- fracture strain, ϵ_C ;
- defect depth, based on the amount of inspection, E ;
- applied load, K, L ; and
- variability in the S-N curves.

For the material properties, the normal distribution was found appropriate. Both the long-term cyclic tension and the maximum line tension were modelled using 3- and 2-parameter Weibull distributions respectively, **Table A-1** and **Table A-2**.

Table A-1

Parameters in Weibull Distribution of Long Term Cyclic Line Tension

$$F(\delta LT) = 1 - \exp \left(- \left(\frac{\delta LT}{\delta LT_c} \right)^{\gamma_\delta} \right)$$

Water Depth (m)	δLT_c (kN)	γ_δ
60	30.0	.626
100	19.6	.670
150	12.8	.641
300	4.1	.591
$\delta LT =$ change in line tension (applied load range).		

TABLE A-2

Maximum Line Tension, Weibull Distribution

$$F(LT_{\max}) = 1 - \exp\left(-\left(\frac{LT_{\max} - LT_o}{LT_c - LT_o}\right)^\gamma\right)$$

Water Depth (m)	LT_o (kN)	$LT_c - LT_o$ (kN)	γ
60	725	7.38	.506
100	800	6.47	.525
150	906	7.86	.564
300	1175	4.23	.581

LT = line tension

For the CTOD design criterion, a full-size specimen CTOD design curve was derived

$$\frac{\delta_c}{2\pi a_c e_y} = 0.5 \left(\frac{e_y}{e_y}\right)^{1.2}$$

where e_y is the yield strain;
 δ_c is the fracture toughness; and
 a_c is the critical defect size.

This design curve does not include a safety factor, since it follows very closely the test data, Figure A-2.

The probability of a flaw detection was expressed as (Figure A-3)

$$P_d(a > a_m) = 1 - e^{-\alpha a_m}$$

where a_m = minimum defect size.

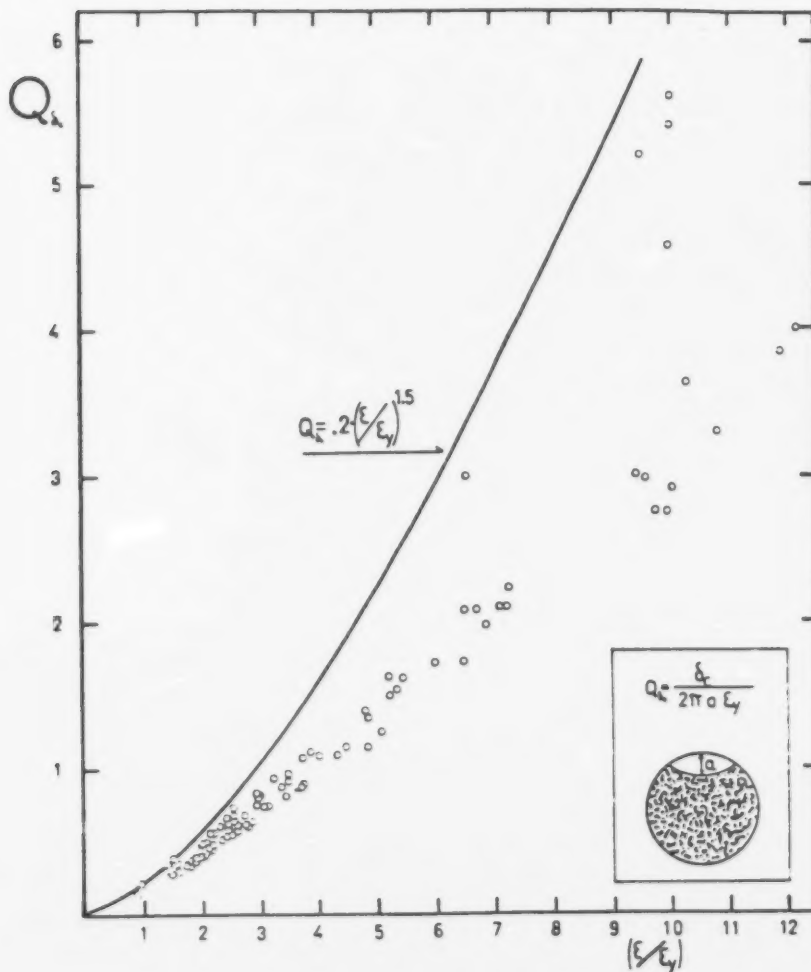


Figure A-2: δ_c = Chain Design Curve for ORQ Chain

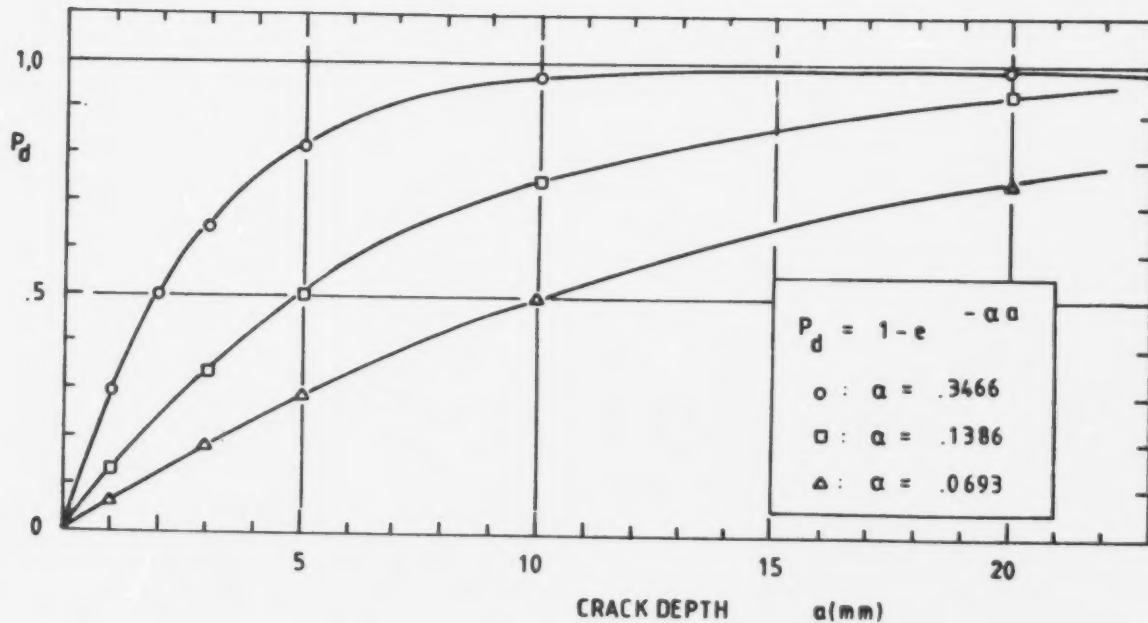


Figure A-3: P_d , Detection Probability with Variation in Crack Depth, a . Three Randomly Selected Values of α for $P_d = .5$ at 2 mm, 5 mm, and 10 mm Respectively

From the actual ultrasonic inspection (26,801 links inspected), the cumulative distributions of defects were obtained:

$$F(a \leq a_m) = 1 - e^{-4.7(a-0.01)} \quad (\text{bend region})$$

and

$$F(a \leq a_m) = 1 - e^{-(a - 0.01)} \quad (\text{weld zones})$$

The estimated minimum limit in defect size is 0.01 mm which is in the order of the roughness of the surface of the links.

Major Results of the Analysis

The results indicate that the elastic-plastic fracture criterion (CTOD) and the critical strain criterion (ϵ_c) best reflect the real failure probability of the chain links. This was confirmed by the data given in Table A-3 for a chain of 4500 links. The failure probability for the various criteria are compared with the real failure rates.

TABLE A-3
Calculated Failure Probability Compared with
Real Life Failure Rate

Failure Criterion	ANALYSIS Years of Service		REAL LIFE Years of Service 7-8
	0	10	
K_c	.98	1.0	FAILURE RATE $\frac{\text{No. of broken chains}}{\text{Tot. No. of chains}} = 8.7 \times 10^{-2} - 11.6 \times 10^{-2}$
δ_c	3.3×10^{-2}	12.6×10^{-2}	
ϵ_c	1.7×10^{-2}	9×10^{-2}	
L_c	4.5×10^{-7}	1.4×10^{-2}	

NOTES:

One chain $\approx$ 4500 links

Number of broken chains, Oil Rig Quality, per Jan 1982: $\approx$ 30-40.

Total number of chains in service: $8 \times 43 = 344$.

Load distribution for 150 m water depth.

Some of the graphical output of the probabilistic analysis is given in Figures A-4, A-5, A-6 and A-7.

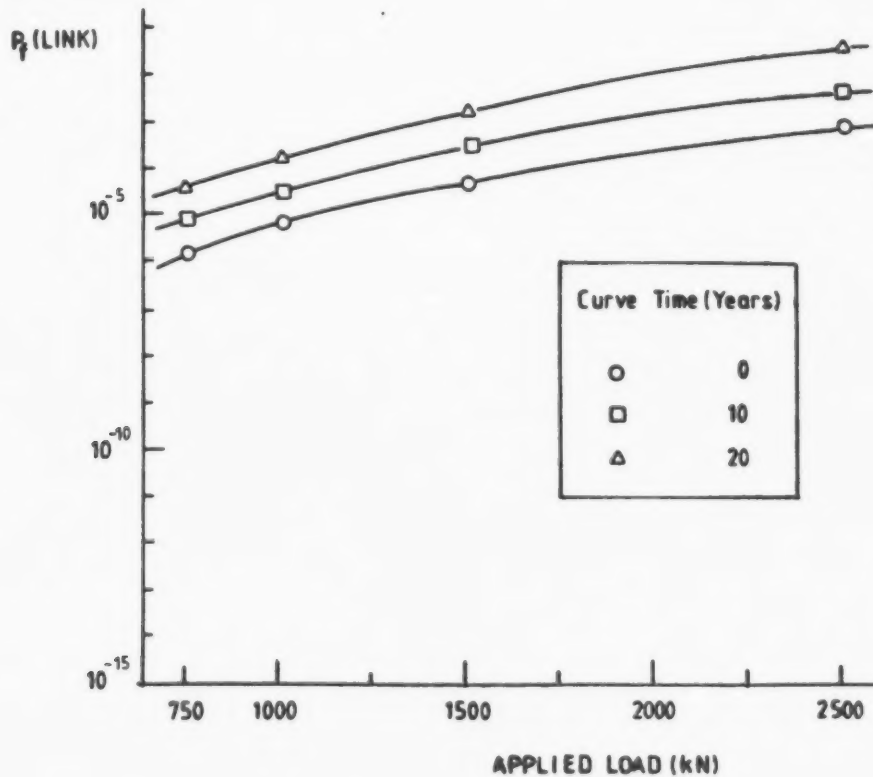


Figure A-4: Failure Probability vs. Applied Load for δ_c -based Failure Criterion. Pure Tension Loading, Loadcase 1

Based on the analysis, it was found that the following three factors govern the failure probability of chains in that order:

1. stress level;
2. initial distribution of defects; and
3. fracture toughness.

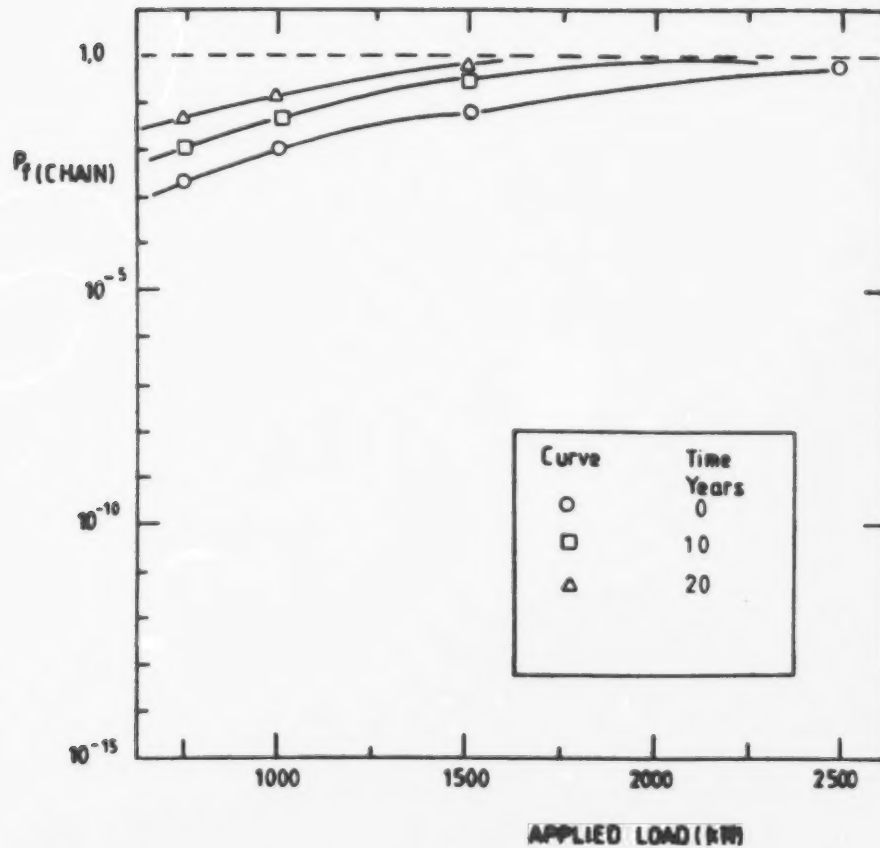


Figure A-5: Failure Probability of a Chain vs. Applied Load
 δ_c -Based Failure Criterion, ORQ-data

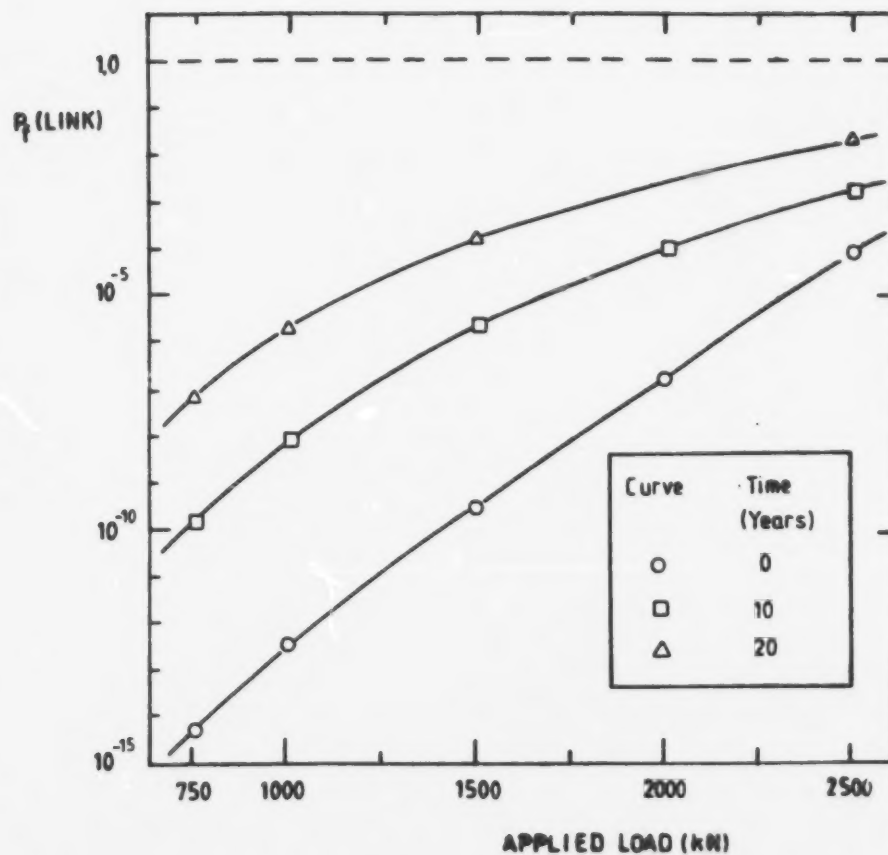


Figure A-6: Failure Probability of a Single Link vs. Applied Load. Tension/Bending Loading. Plastic Collapse Failure Criterion. Limit Load Criterion

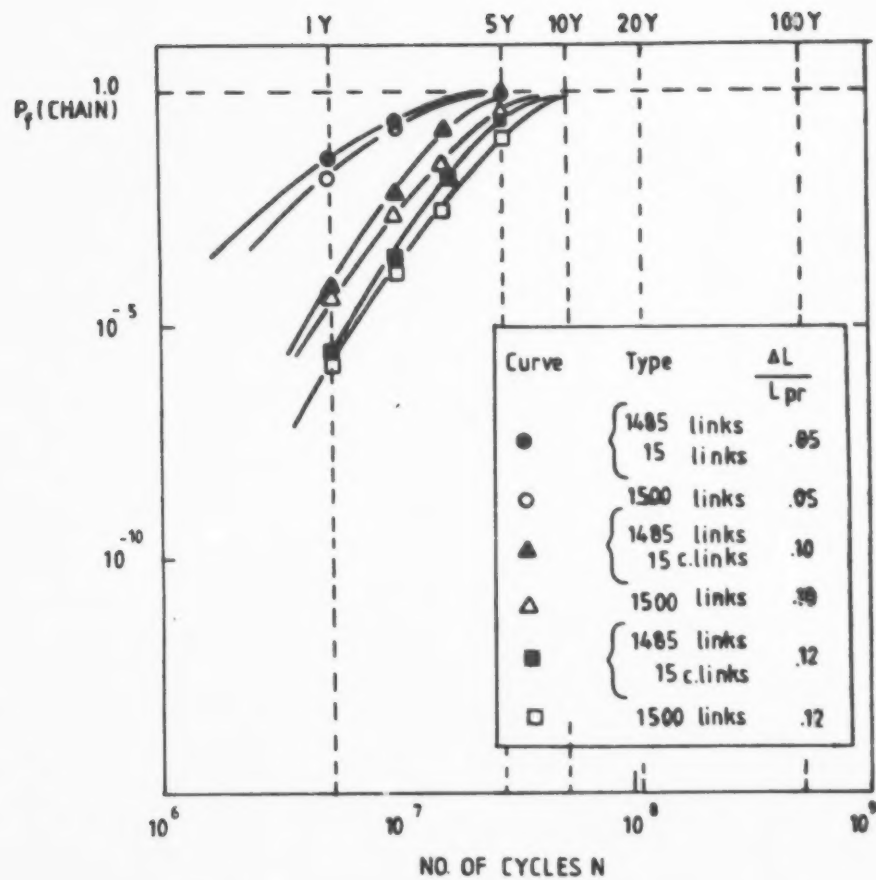
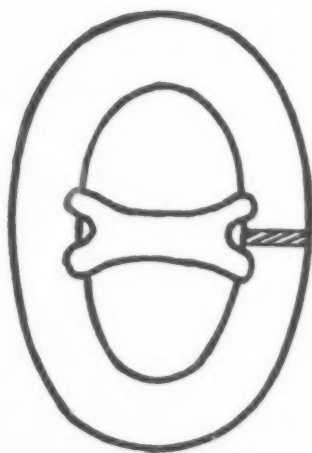


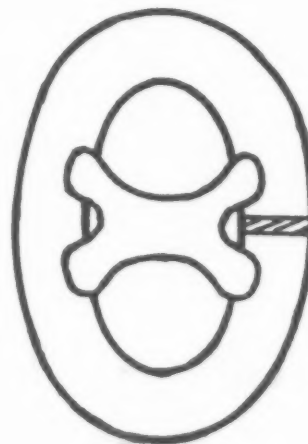
Figure A-7: Failure Probability of a Chain for Palmgren-Miner Analysis, 1500 Links

A reduction of the initial defect size seems highly attractive, however it is hard to achieve in practice due to uncertainties inherent in the ultrasonic inspection methods. Still, an alternative study design, **Figure A-8**, leads to a better access for the inspection of the flash built weld thus improving the probability of detection.

Fracture toughness is the most favorable parameter to modify with the aim of improving the chain reliability. This may be achieved through a more stringent quality control in the entire fabrication process including the steel-making and delivery.



a) New design



b) New design with extra stability

Figure A-8: Alternate Chain Link Design

CASE 2 LNG TANKS - MOSS ROSENBERG DESIGN

The Moss Rosenberg LNG design, **Figure A9**, was introduced in 1970. The ships built to this design show an excellent safety record. In view of this record, it was of a considerable interest to reevaluate probabilities of failure at critical locations and relate the sensitivity of these probabilities to various basic quantities (Fredheim, 1983).

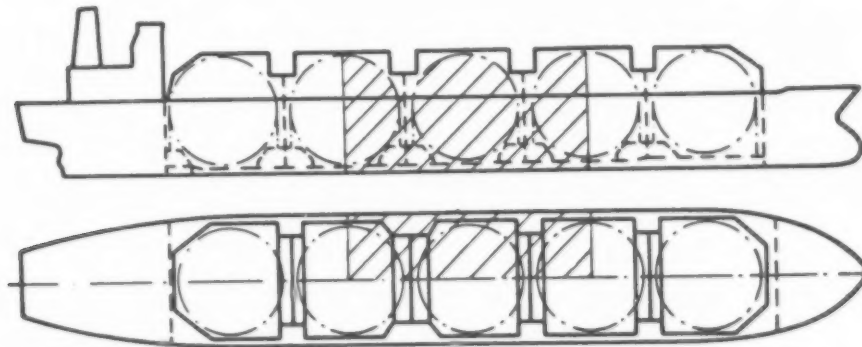


Figure A9: Moss Rosenberg Design of LNG Tanker

Two materials have been used in the design: 9% Ni steel and aluminium alloy AA 5083. Both materials preserve high ductility down to the service temperature of -162°C , which is the temperature of a liquid natural gas. Both materials were extensively tested for fracture toughness, fatigue crack propagation and buckling to gain the necessary information about material properties.

The following failure criteria were used in the analysis:

- buckling;
- crack propagation (both Palmgren-Miner and Paris-Erdogan); and
- unstable fracture.

The stress analysis highlighted two critical locations:

- 1) connection between the tank shell and the equatorial profile in the lower hemisphere; and
- 2) connection between the tank shell and the tower at the bottom of the tank.

The following loads were considered:

- i) Static loads:
 - tower weight;
 - internal overpressure (0.25 bar);
 - weight of the tank; and
 - cargo weight.
- ii) Dynamic loads:
 - vertical acceleration of the tower (0.5 g);
 - vertical acceleration of the tank material (0.5 g);
 - vertical acceleration of the cargo (0.5 g); and
 - sloshing.

Data from newbuildings of ships carrying spherical LNG tanks was used to establish the typical defect density distributions in the tank welds before and after inspection/repair.

Discussion of the Results

Figure A-10 shows the probability of the buckling failure as a function of the plate thickness and the imperfection amplitude. Strong sensitivity of P_f to both of these parameters is evident. Figure A-11 shows the effect of the extent of NDE control on the probability of the fatigue failure (through-thickness crack propagation). It should be noted that the gain in improved reliability is minimal when increasing the amount of the NDE control beyond 50%.

- A16 -

9% NI-STEEL BUCKLING

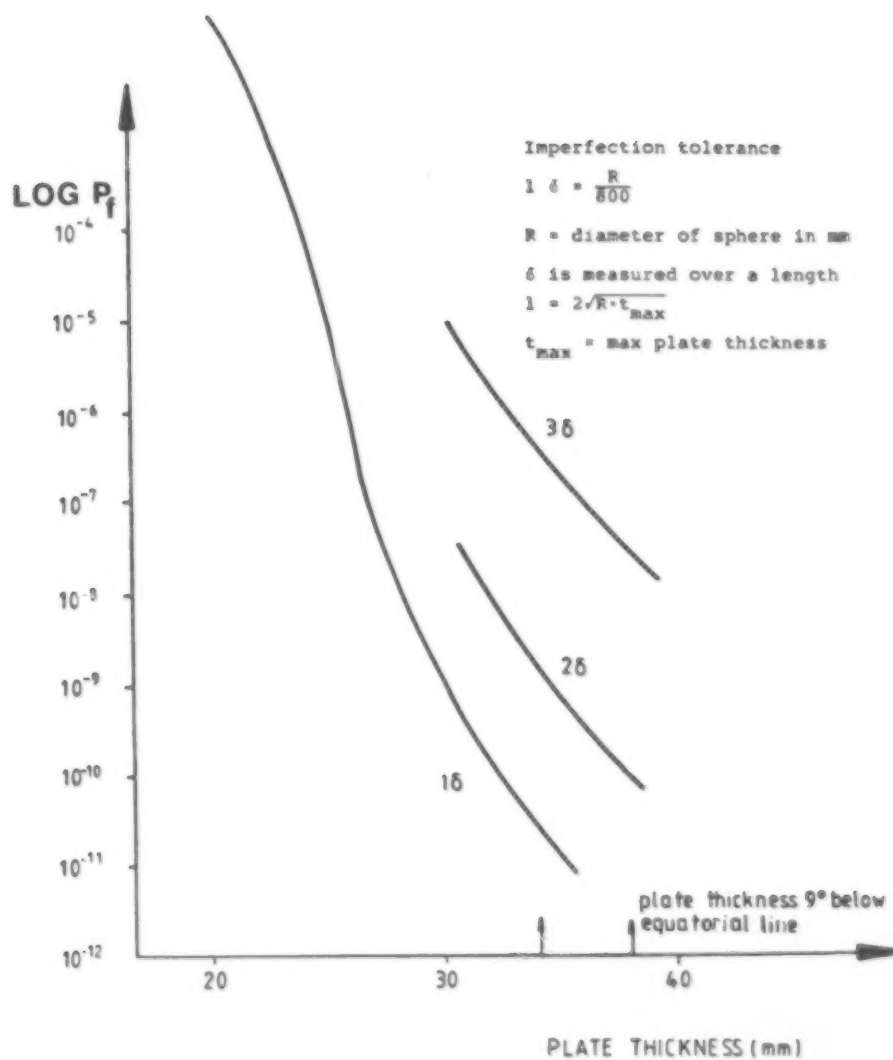


Figure A-10: Failure Analysis Based on a Distribution of Imperfections

$$\sigma_{\phi}/\sigma_{\theta} = -1$$

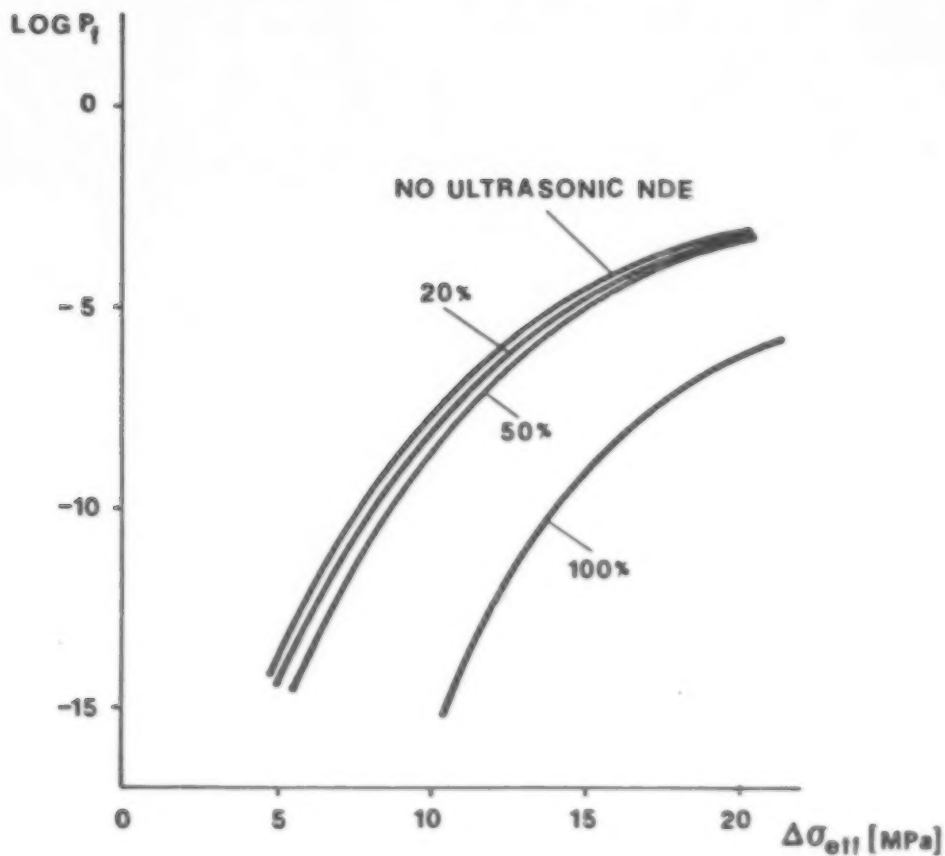


Figure A-11: Effect of NDE on Probability of Failure

Figure A-12 shows an interesting comparison of the P_f curves for surface and internal defects respectively. Below a certain stress threshold, the probability of a fatigue failure associated with internal defects is actually higher than that associated with the surface defects. Obviously the situation would profoundly change with predominating bending stresses. Finally, Figures A-13 and A-14 show a marked difference between probabilities of unstable fracture in the parent material and the heat affected zone (HAZ) respectively.

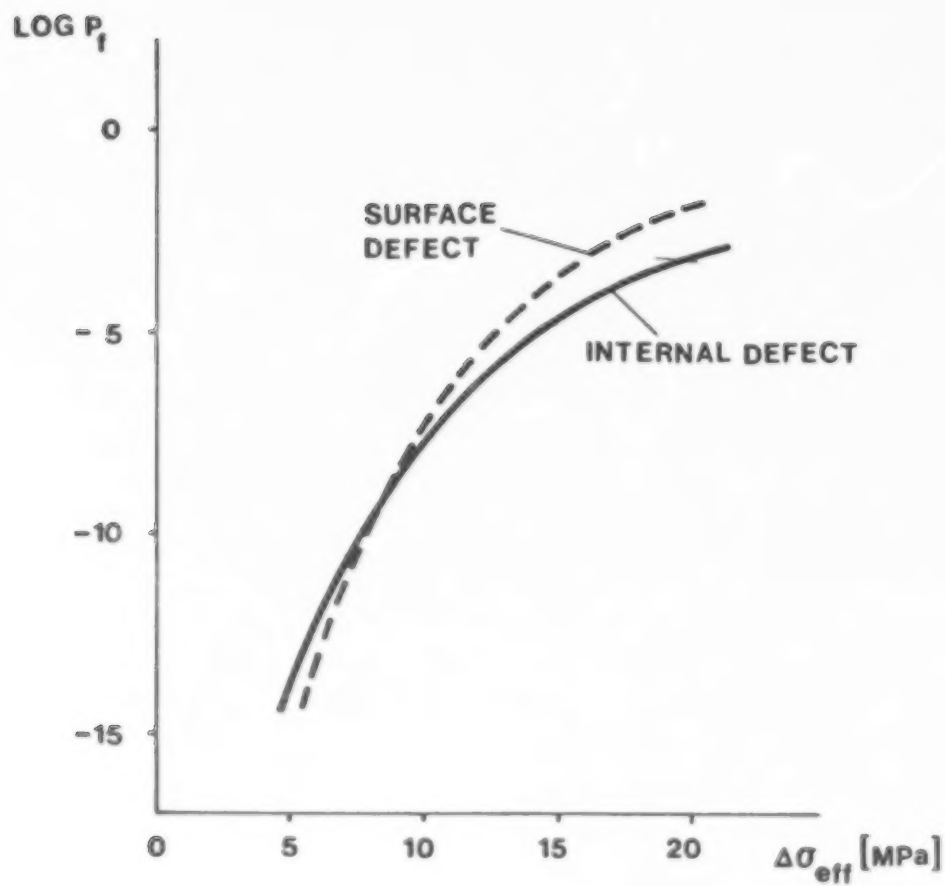


Figure A-12: Comparison of Propagation of Surface and Internal Defects

It can be concluded that the low probability of failure (below 10^{-7}) for all relevant loading cases agrees well with the safety record of the Moss Rosenberg LNG ship design.

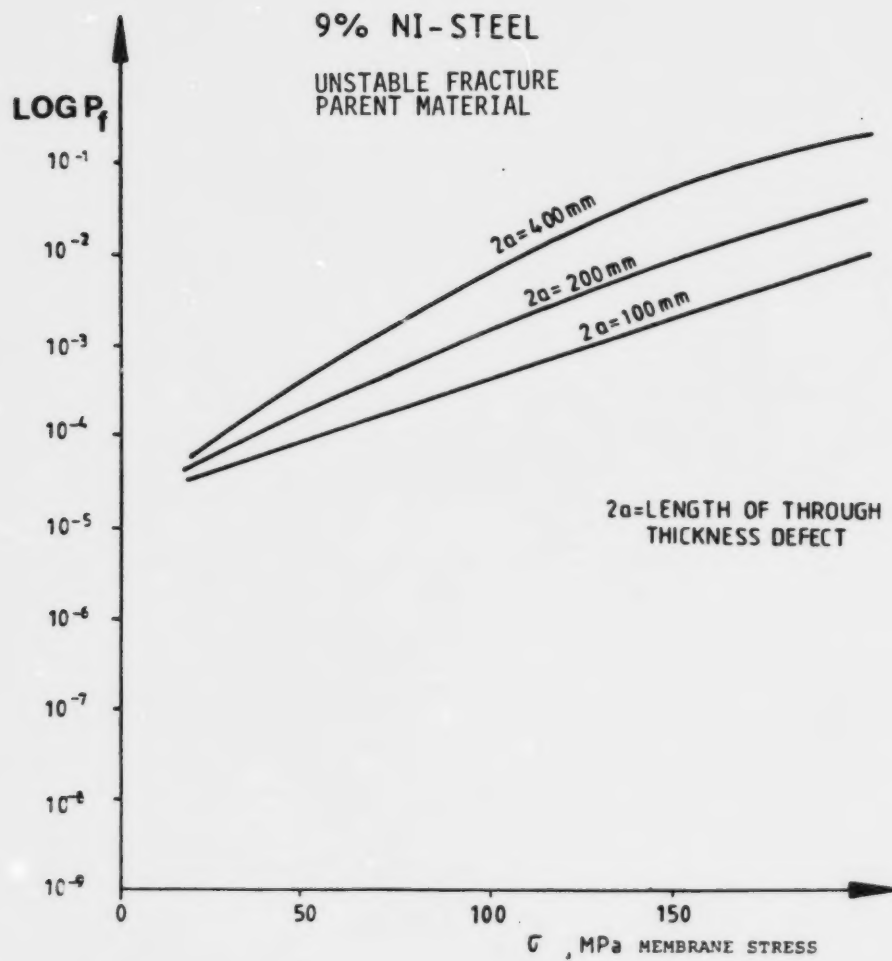


Figure A-13: Probability of Unstable Fracture in Parent Steel

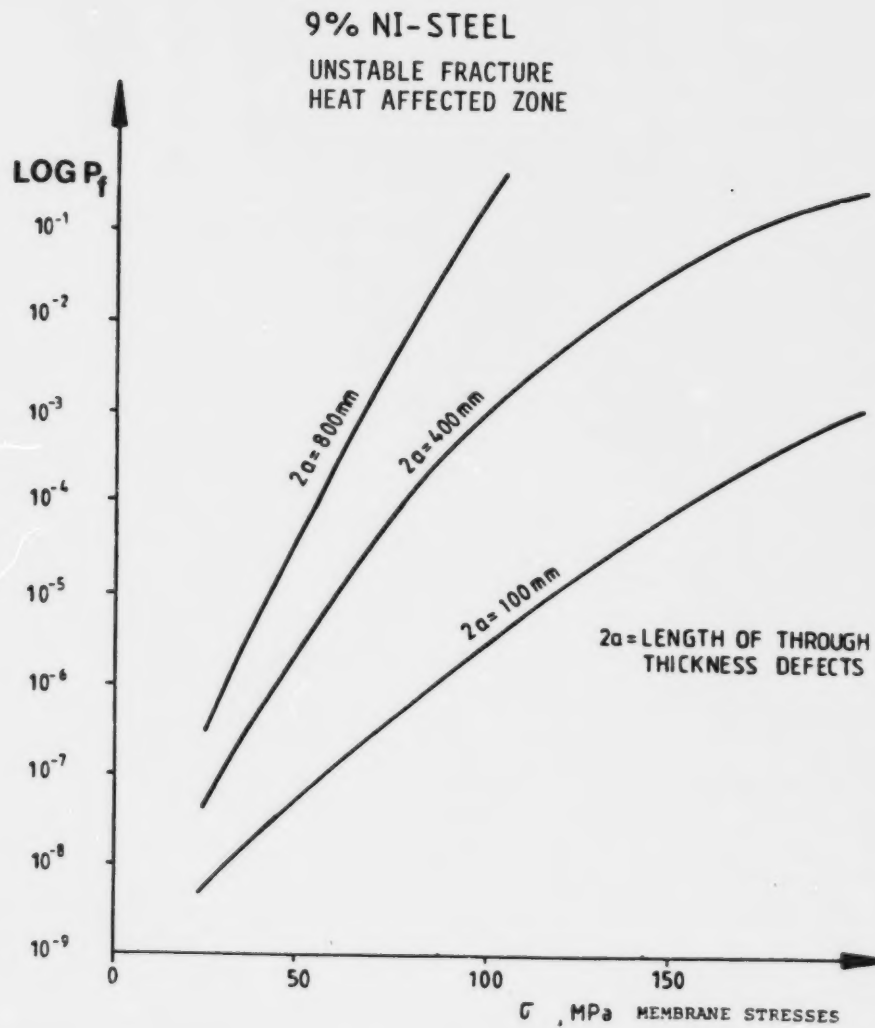


Figure A-14: Probability of Unstable Fracture in Heat Affected Zone

CASE 3 EXTENDED LIFETIME ANALYSIS

As the offshore structures in the North Sea will get closer to the end of their designed (expected) lifetime, an analysis, based on which a life extension could be made, will become a valuable tool (Madsen, Skjong and Zaden, 1986). The decision whether or not to extend the lifetime of a structure will have to be taken shortly before the anticipated lifetime expires. This decision will be affected by the experience gained during the service life of the structure. The probabilistic analysis seems to be best qualified for this type of a decision.

In the following case, a tubular joint of a jacket structure is considered. The anticipated design life was 20 years and the joint performed successfully for the past 15 years without any repair or detectable damage. An extension of the lifetime to 50 years was sought.

First, a reliability analysis was carried out for the original design specifications. **Figure A-15** shows the reliability index as a function of the lifetime. With the requirement of $\beta \geq 2.00$ for the design life, the joint is accepted for a lifetime of 20 years, but not for a lifetime of 50 years. The failure probability after 50 years is

$$P_{50} = P(g(\underline{Z}, 50) \leq 0) = 2.59 \cdot 10^{-2}$$

corresponding to a reliability index

$$\beta_{50} = 1.944$$

Second, it was checked whether the fact that the joint has survived 15 years of service provided enough additional information for a sufficient increase of the reliability. The updated failure probability after 50 years becomes:

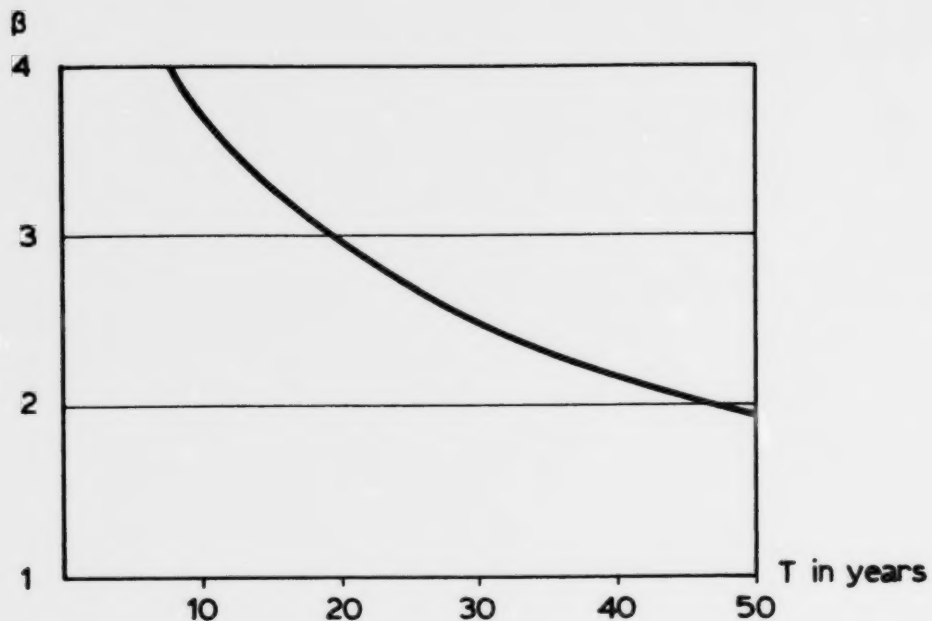


Figure A-15: Reliability Index as a Function of Time

$$P(g(\underline{Z}, 50) \leq 0 | g(\underline{Z}, 15) > 0) = \frac{P(g(\underline{Z}, 50) \leq 0) - P(g(\underline{Z}, 15) \leq 0)}{1 - P(g(\underline{Z}, 15) \leq 0)}$$

using that g decreases with T . An analysis for $T = 15$ gives

$$P_{15} = P(g(\underline{Z}, 15) \leq 0) = 2.53 \cdot 10^{-2}$$

The updated failure probability and the corresponding reliability index thus become:

$$P'_{50} = P(g(\underline{Z}, 50) \leq 0 | g(\underline{Z}, 15) > 0) = 2.53 \cdot 10^{-2}$$

$$\beta'_{50} = -\Phi^{-1}(P'_{50}) = 1.955$$

The requirement $\beta \geq 2.00$ is therefore not fulfilled with this simple updating.

During the 15 years of service, the environmental conditions and the global response have been measured. The uncertainty in the long term sea state probabilities and the uncertainty in the transfer functions for the element forces are thus reduced and the mean values may be somewhat changed. Repeating the analysis with updated distributions leads to:

$$P_{50}^* = 1.64 \times 10^{-2}, \beta_{50}^* = 2.135$$

$$P_{15}^* = 1.23 \times 10^{-4}$$

$$P_{50}^{*'} = 1.63 \times 10^{-2}, \beta_{50}^* = 2.137$$

The requirement of $\beta \geq 2.00$ is thus fulfilled. Again it is noted that the effect of the information that the joint has survived 15 years of service is of very limited value when the increase in lifetime is as large as in this case.

APPENDIX B

OUTPUT OF PROBAN ANALYSIS

```

*****
*                               SET NUMBER : 14                               *
*                               LIMIT STATE:  0                               *
*****
*  BETA  *  PROB  *  BTDIFF *  ANGDIFF *  ITER *
* .3269D+01*.54063D-03* .9D-05 * .3D-02* 4 *
*****
*  *  VARNR  *  DISTR  *  DES.POINT  *  SENSIT  *  *
*  *    1  *  NRM  *  .1426D+03 * -.1015D+00 *  *
*  *    2  *  NRM  *  .1007D+02 * -.2105D-01 *  *
*  *    3  *  NRM  *  .7115D+00 * -.3515D-01 *  *
*  *    4  *  NRM  *  .3675D+03 * -.7008D-01 *  *
*  *    5  *  NRM  * -.1048D+01 * .3592D+00 *  *
*  *    6  *  NRM  * .9704D-01 * -.8795D+00 *  *
*  *    7  *  NRM  * .4897D-01 * .6291D-01 *  *
*  *    8  *  NRM  * .2397D+01 * .1263D+00 *  *
*  *    9  *  FIX  * .9133D+00 * .0000D+00 *  *
*  *   10  *  NRM  * .5689D-01 * -.1740D-01 *  *
*  *   11  *  NRM  * .1211D+00 * -.3706D-01 *  *
*  *   12  *  NRM  * .1209D+01 * -.2428D+00 *  *
*****

```

LEGEND

SET NUMBER: Load case

LIMIT STATE: Sequential number for the Library of limit state functions (not used)

BETA: Reliability index

PROB: Probability of failure

BTDIFF: Difference $\beta_n - \beta_{n-1}$ at last iteration

ANGDIFF: Angular difference at last iteration

ITER: Total number of iterations along the failure surface

VARNR: Sequential number assigned to a basic quantity (variable)

DISTR: Abbreviated type of distribution used

DES. POINT: Value of basic quantity at the design point

SENSIT: Sensitivity factor α

Figure B1. Typical PROBAN Output

APPENDIX C

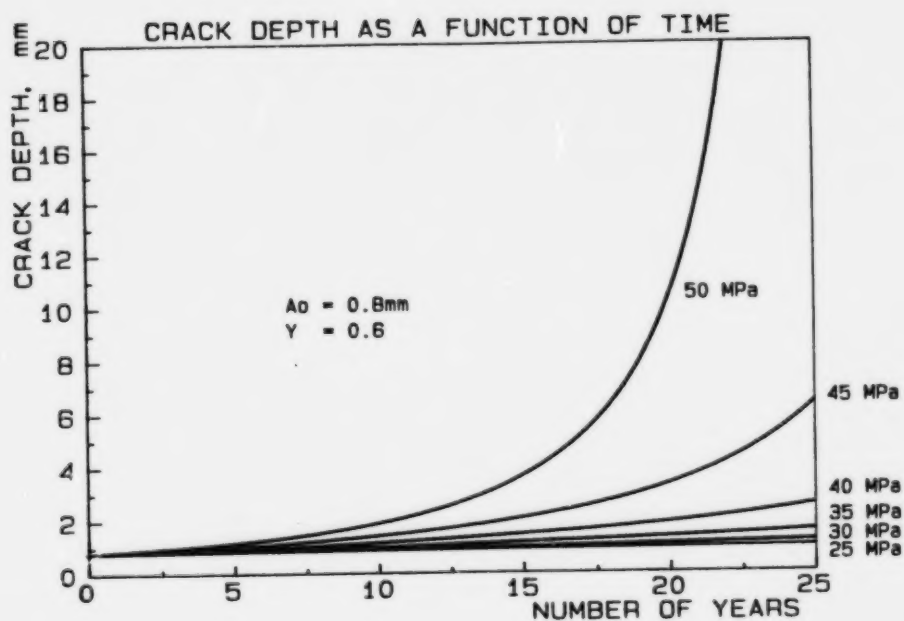


Figure C-1. Crack Depth as a Function of Time

$$A_0 = 0.8 \text{ mm}, Y = 0.6$$

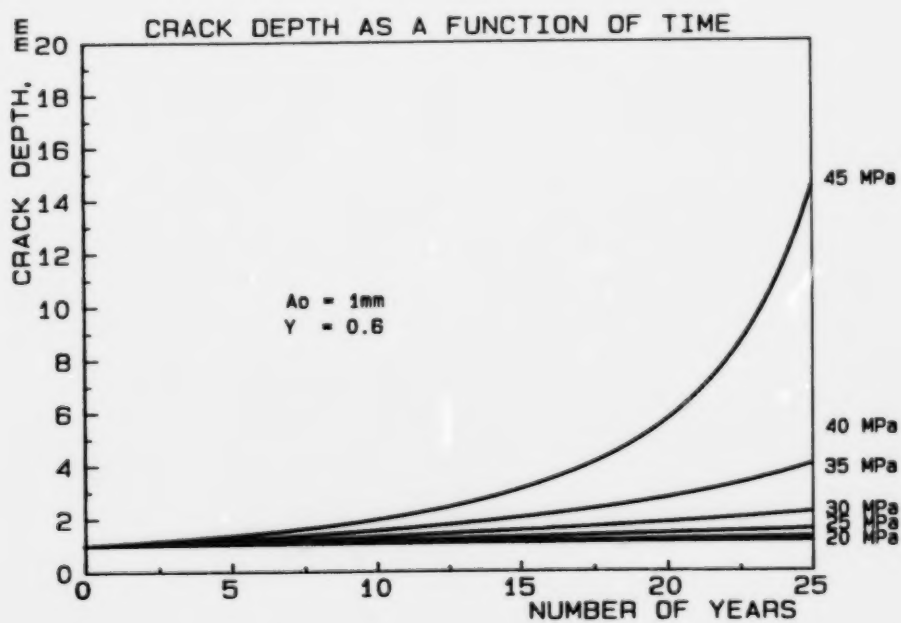


Figure C-2. Crack Depth as a Function of Time

$$A_0 = 1 \text{ mm}, Y = 0.6$$

- C2 -

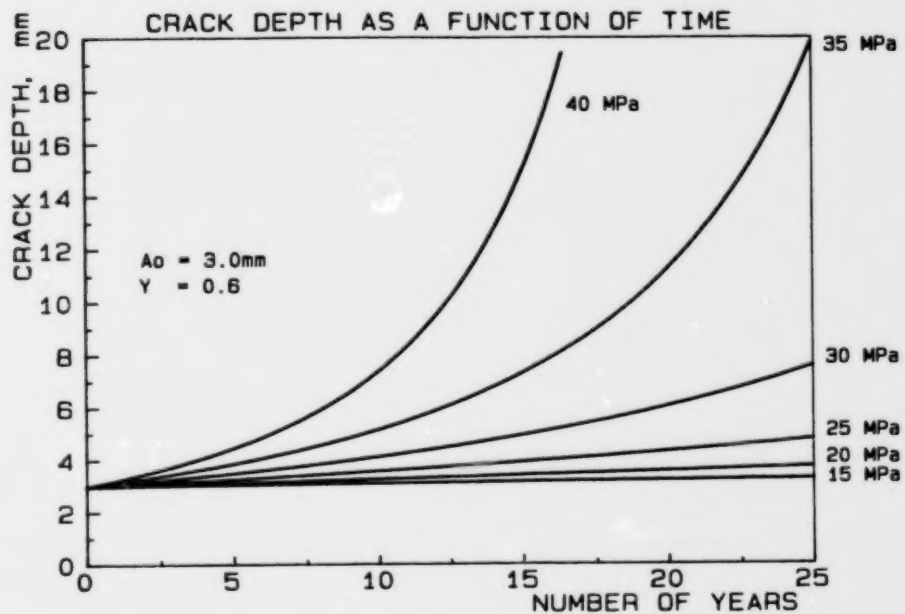


Figure C-3. Crack Depth as a Function of Time

$$A_0 = 3 \text{ mm}, Y = 0.6$$

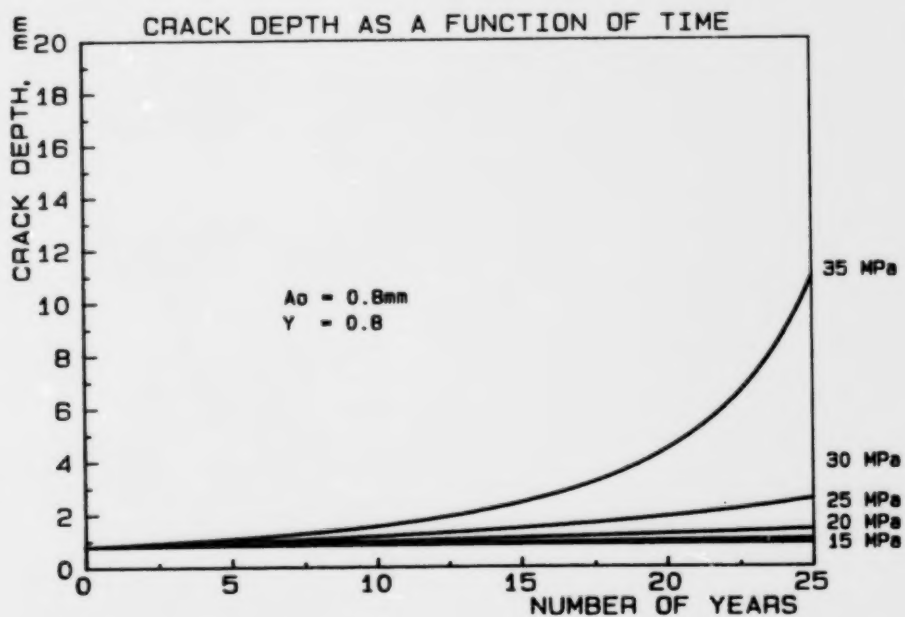


Figure C-4. Crack Depth as a Function of Time

$$A_0 = 0.8 \text{ mm}, Y = 0.8$$

- C3 -

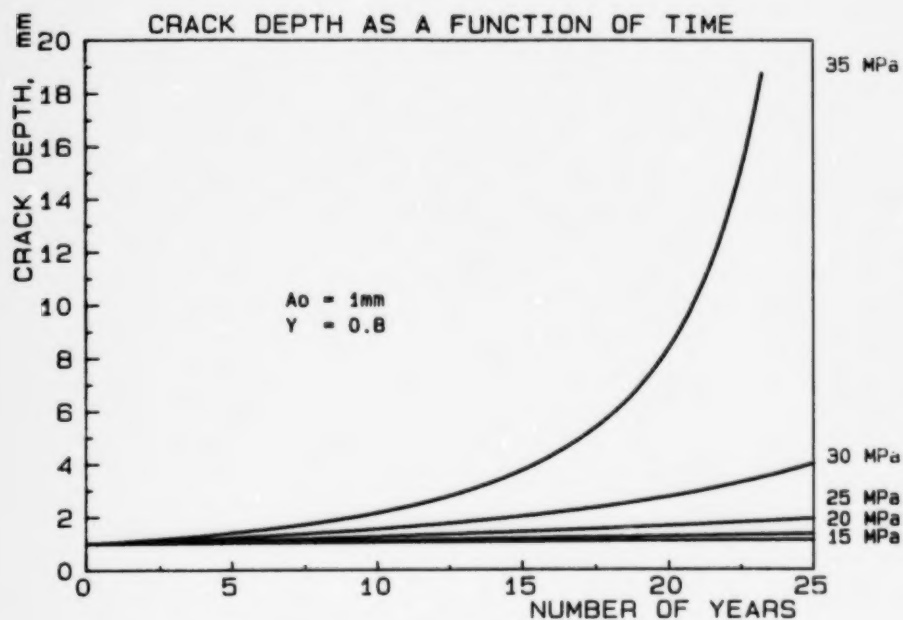


Figure C-5. Crack Depth as a Function of Time

$A_0 = 1\text{ mm}, Y = 0.8$

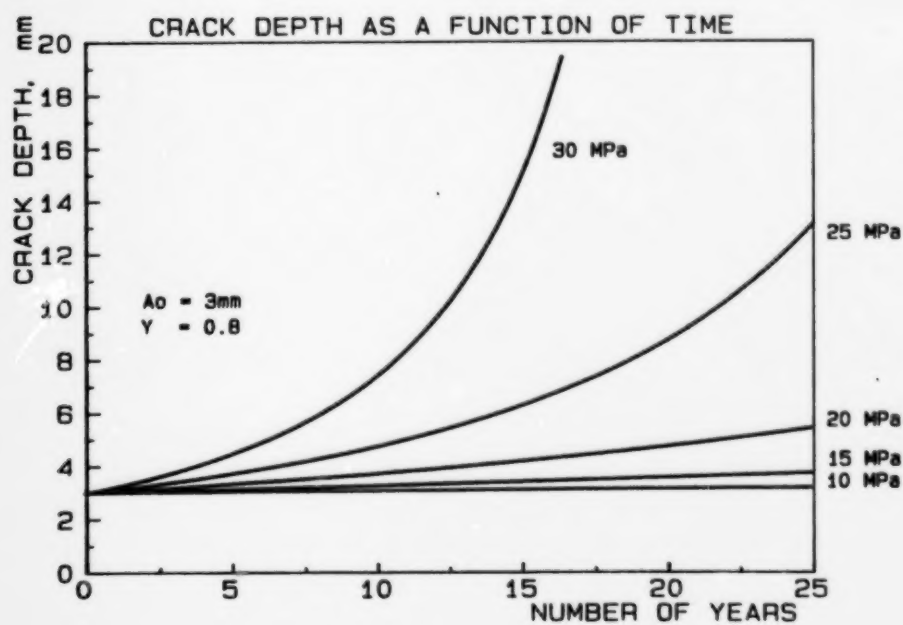


Figure C-6. Crack Depth as a Function of Time

$A_0 = 3\text{ mm}, Y = 0.8$

- C4 -

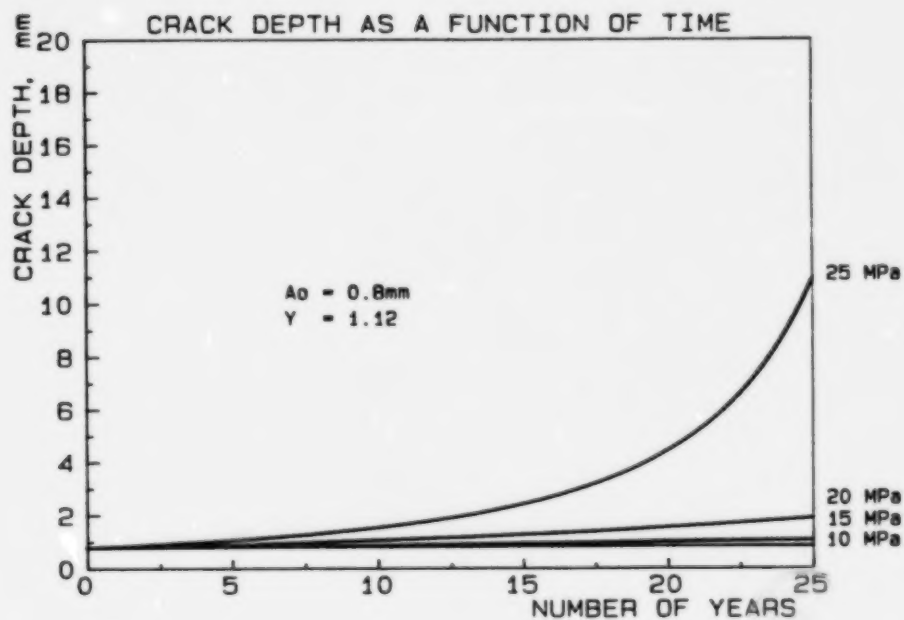


Figure C-7. Crack Depth as a function of Time

$A_0 = 0.8 \text{ mm}$, $Y = 1.12$

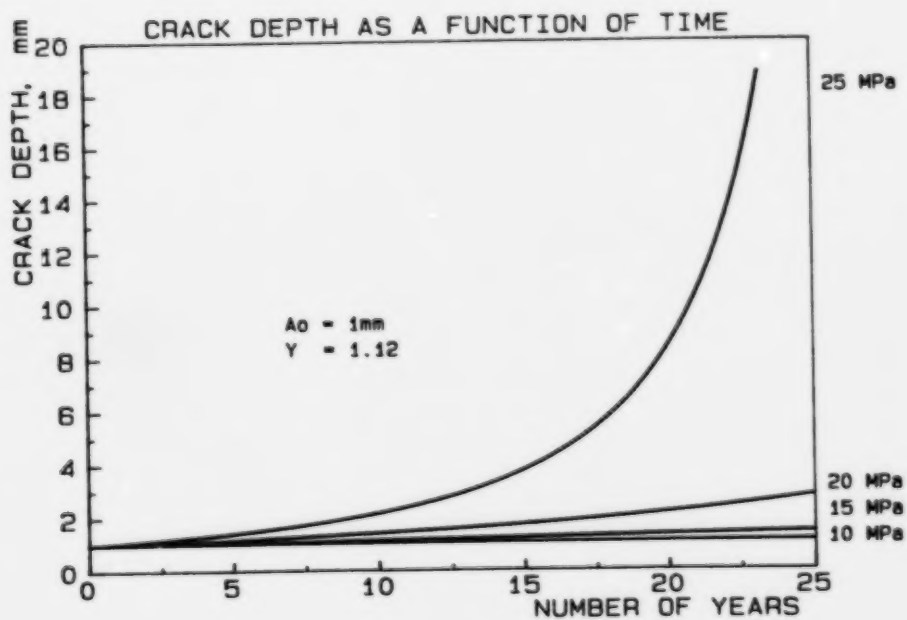


Figure C-8. Crack Depth as a Function of Time

$A_0 = 1 \text{ mm}$, $Y = 1.12$

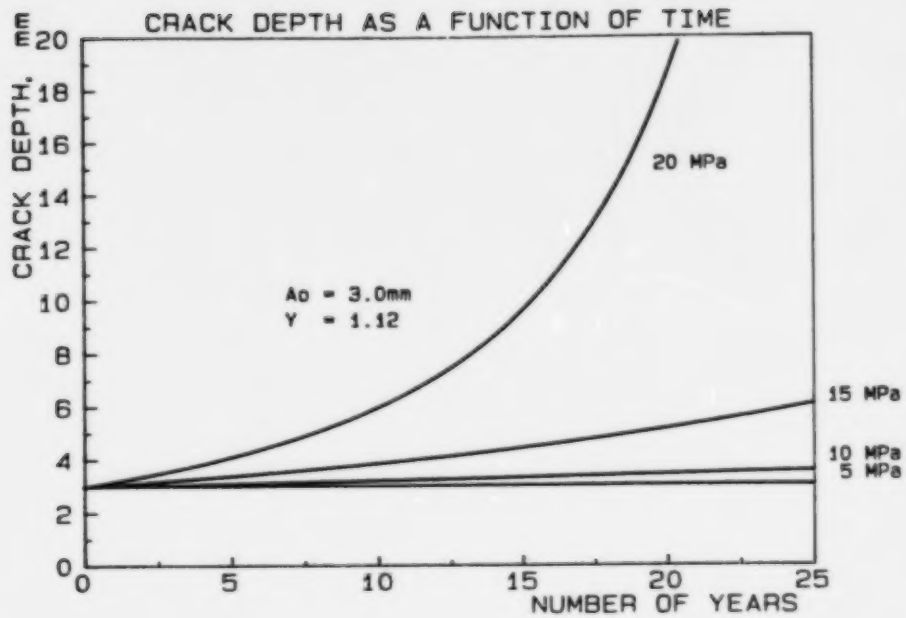


Figure C-9. Crack Depth as a Function of Time
 $A_0 = 3.0 \text{ mm}$, $Y = 1.12$

- C6 -

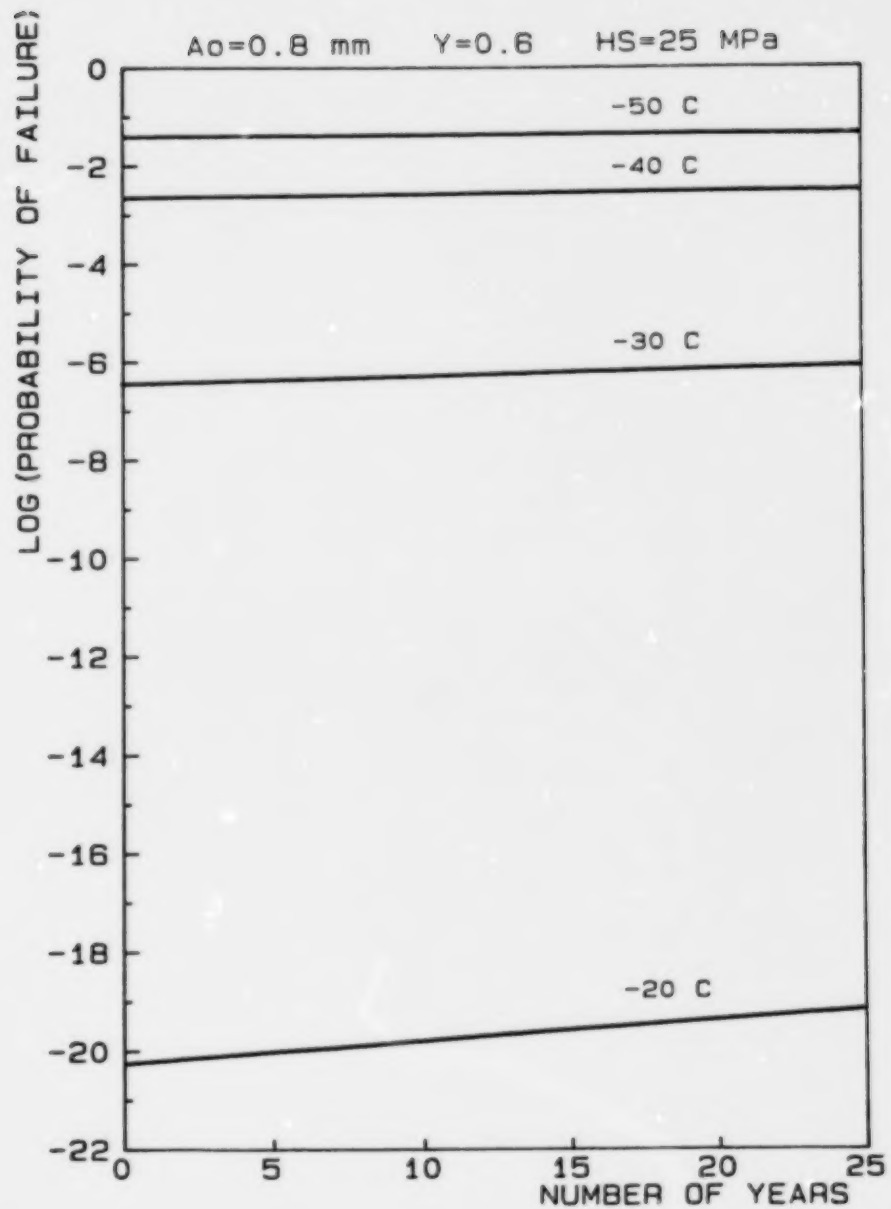


Figure C-10 to C-54

Probability of Failure as a Function of Time

T-Joint, $\Phi = 0.05$

- C7 -

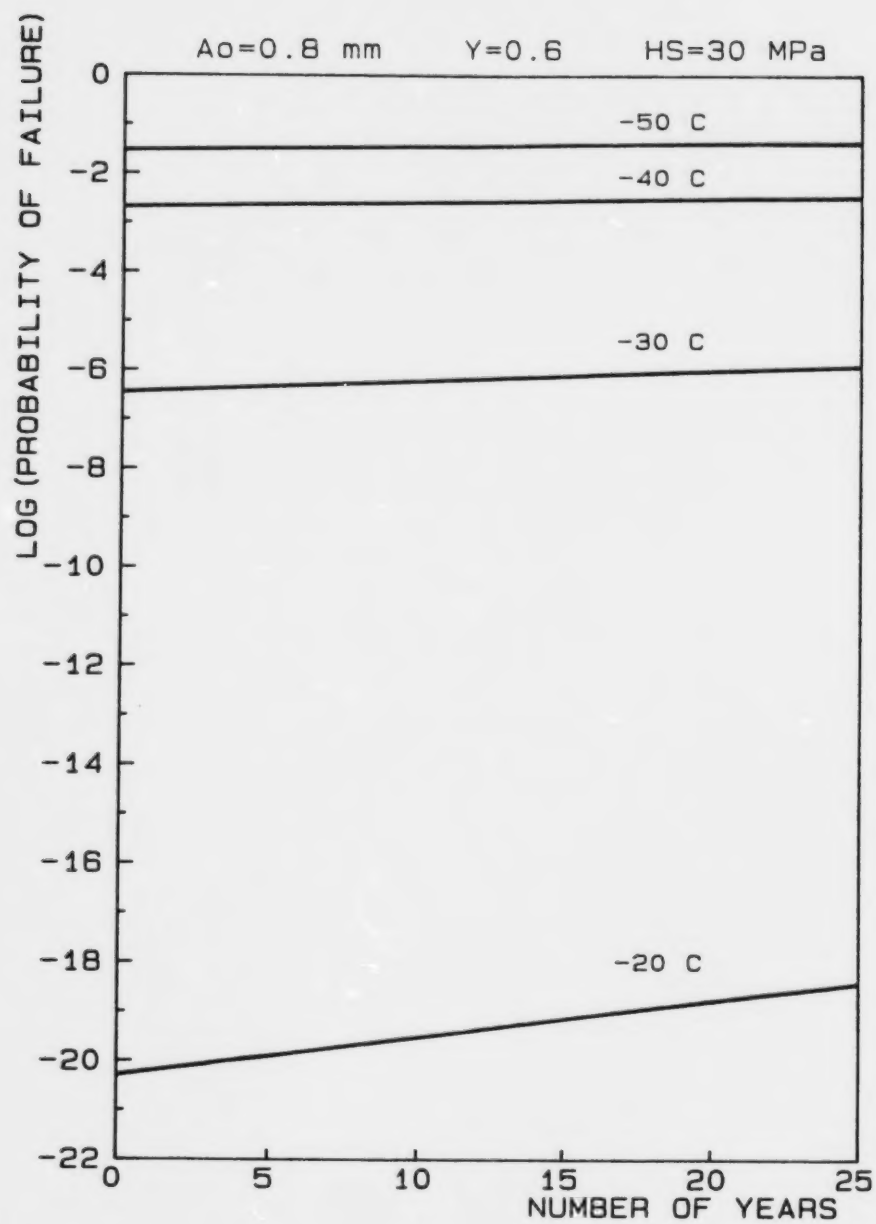


Figure C-11

- C8 -

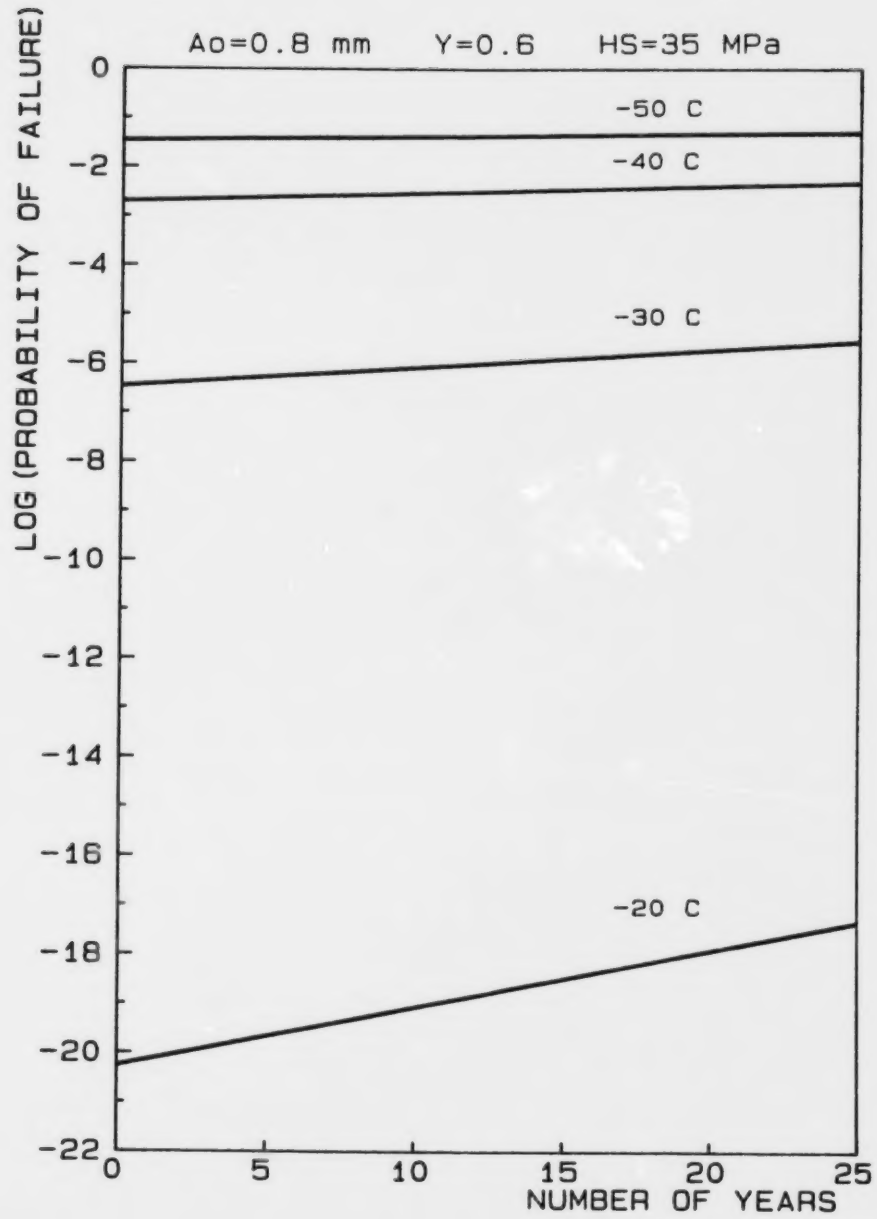


Figure C-12

- C9 -

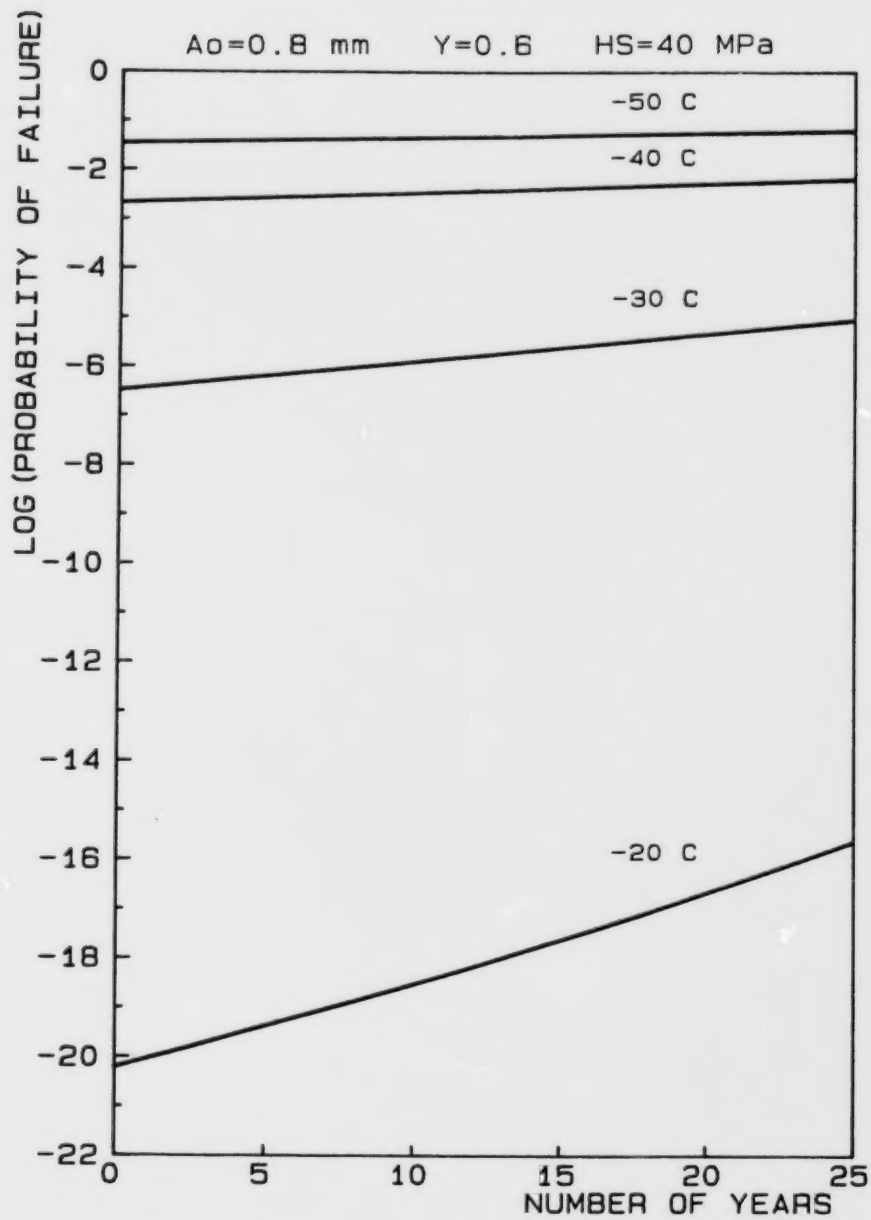


Figure C-13

- C10 -

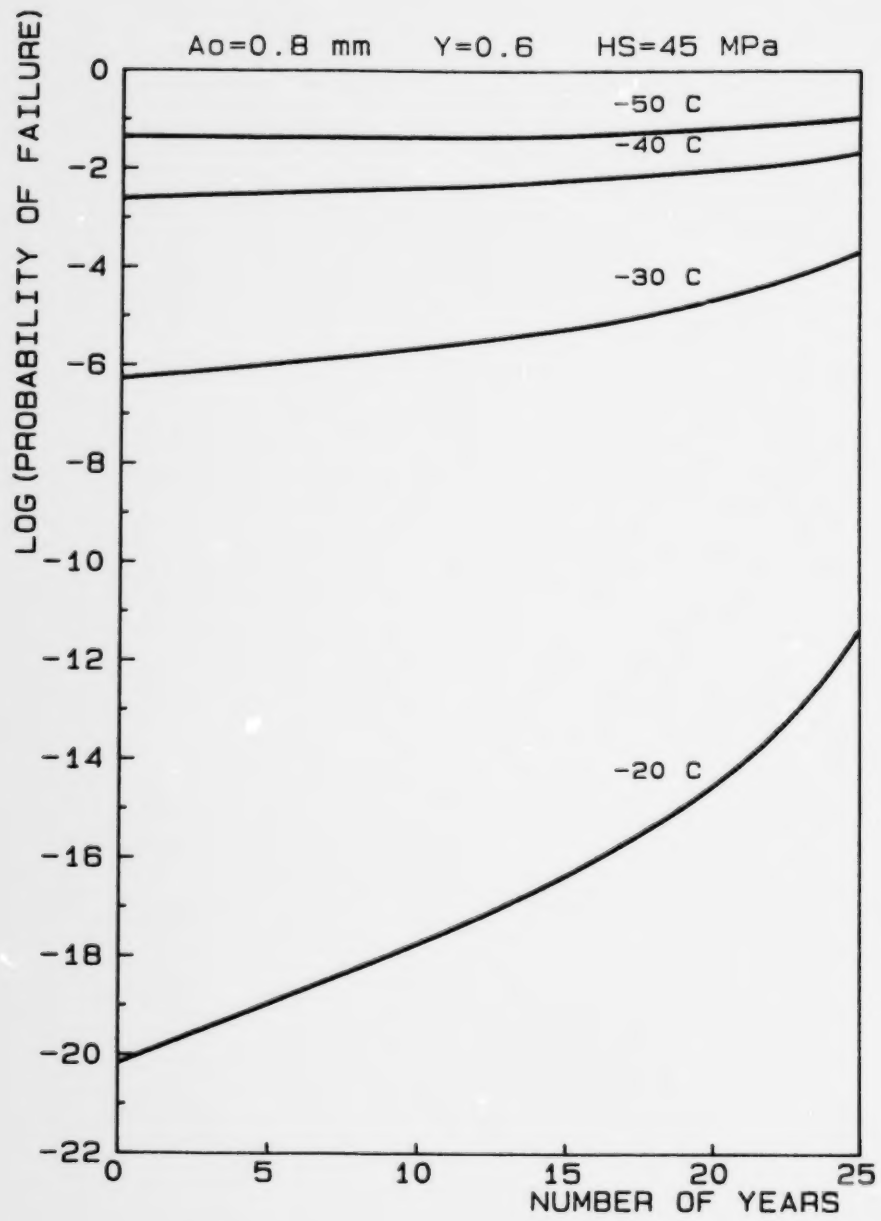


Figure C-14

- C11 -

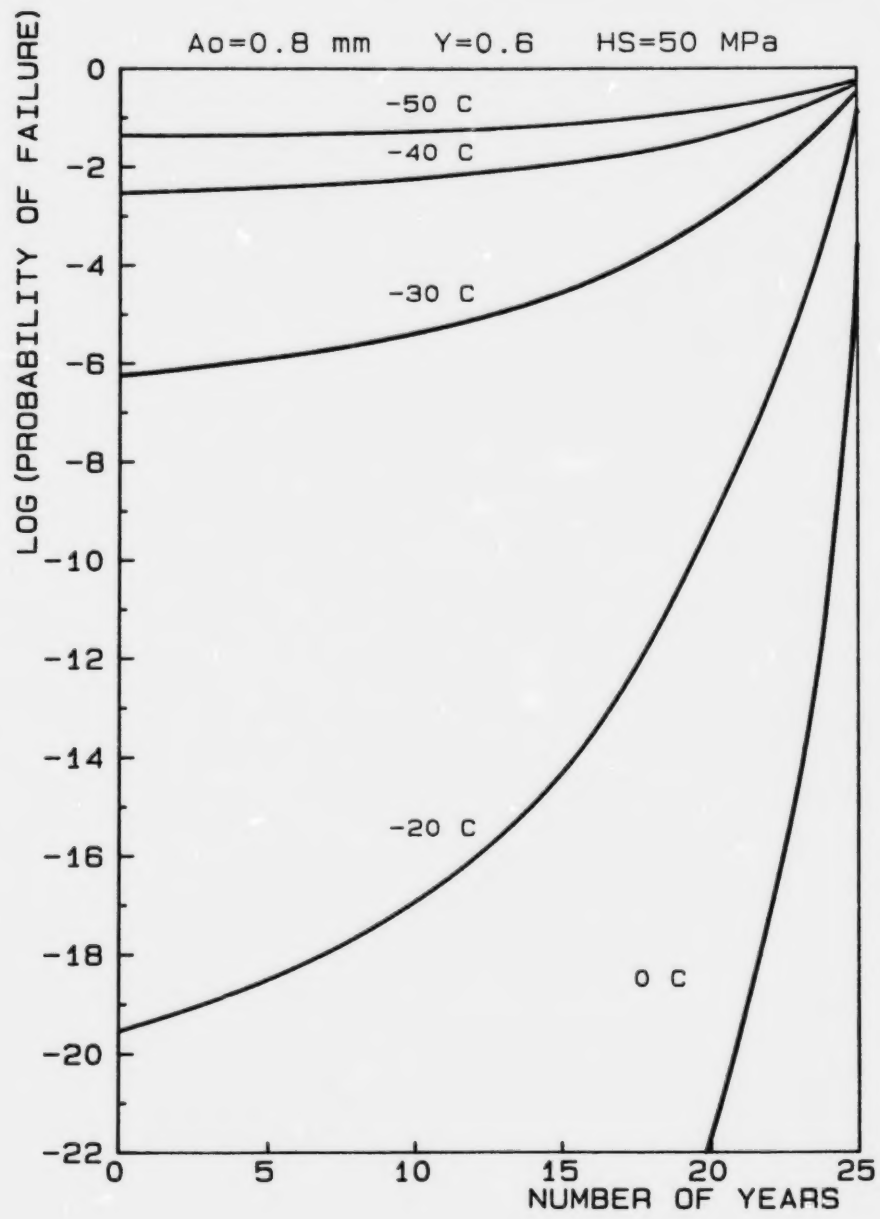


Figure C-15

- C12 -

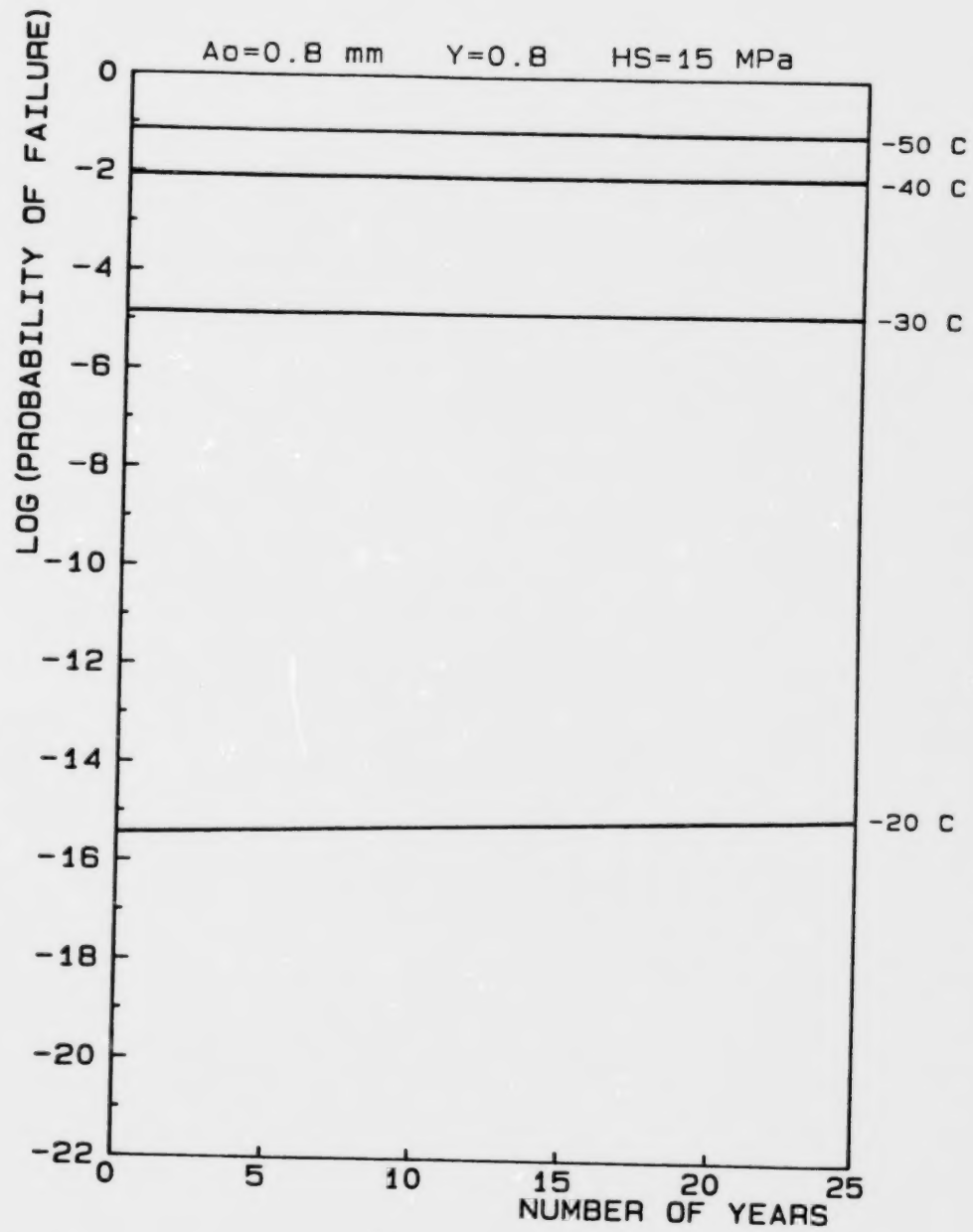


Figure C-16

- C13 -

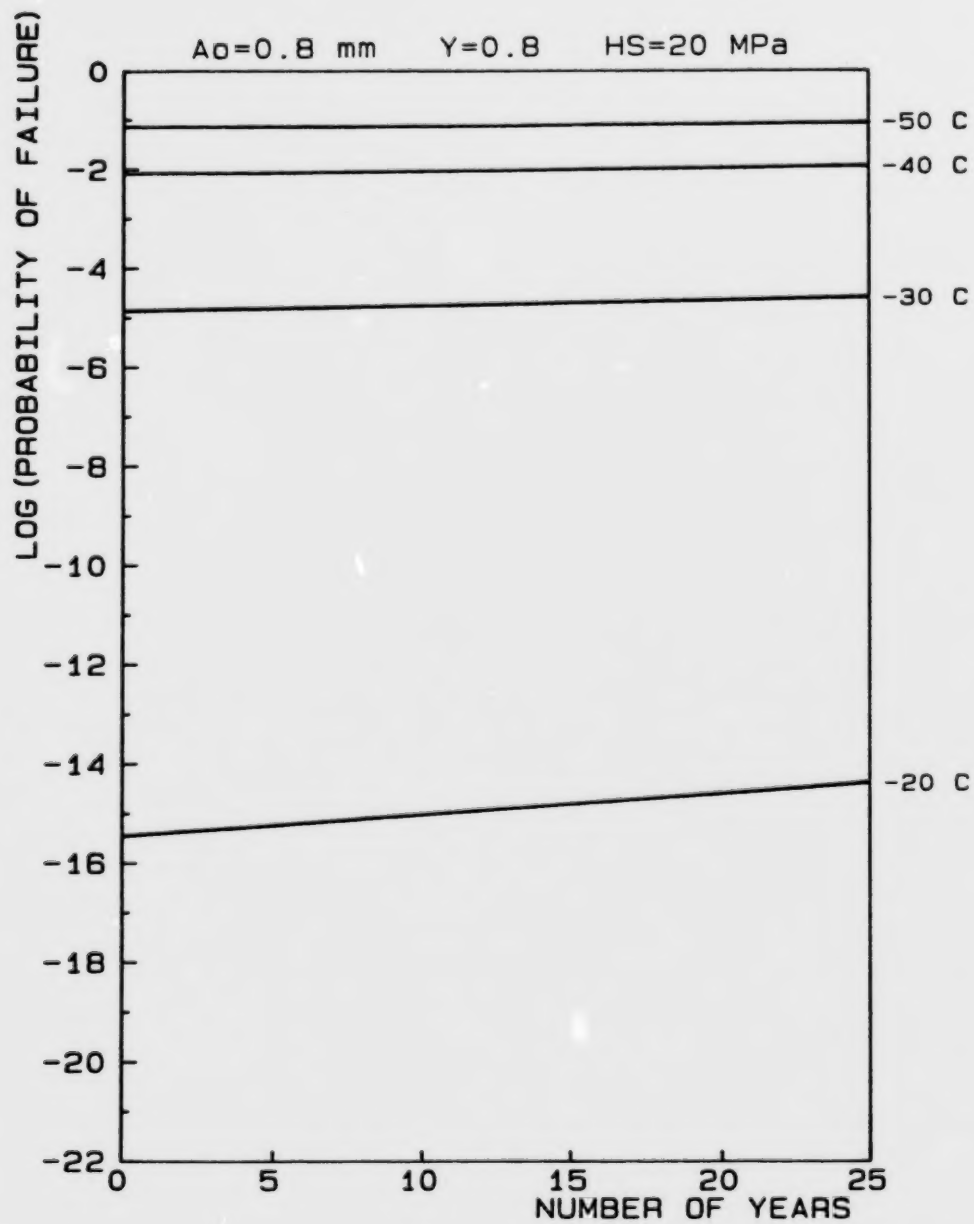


Figure C-17

- C14 -

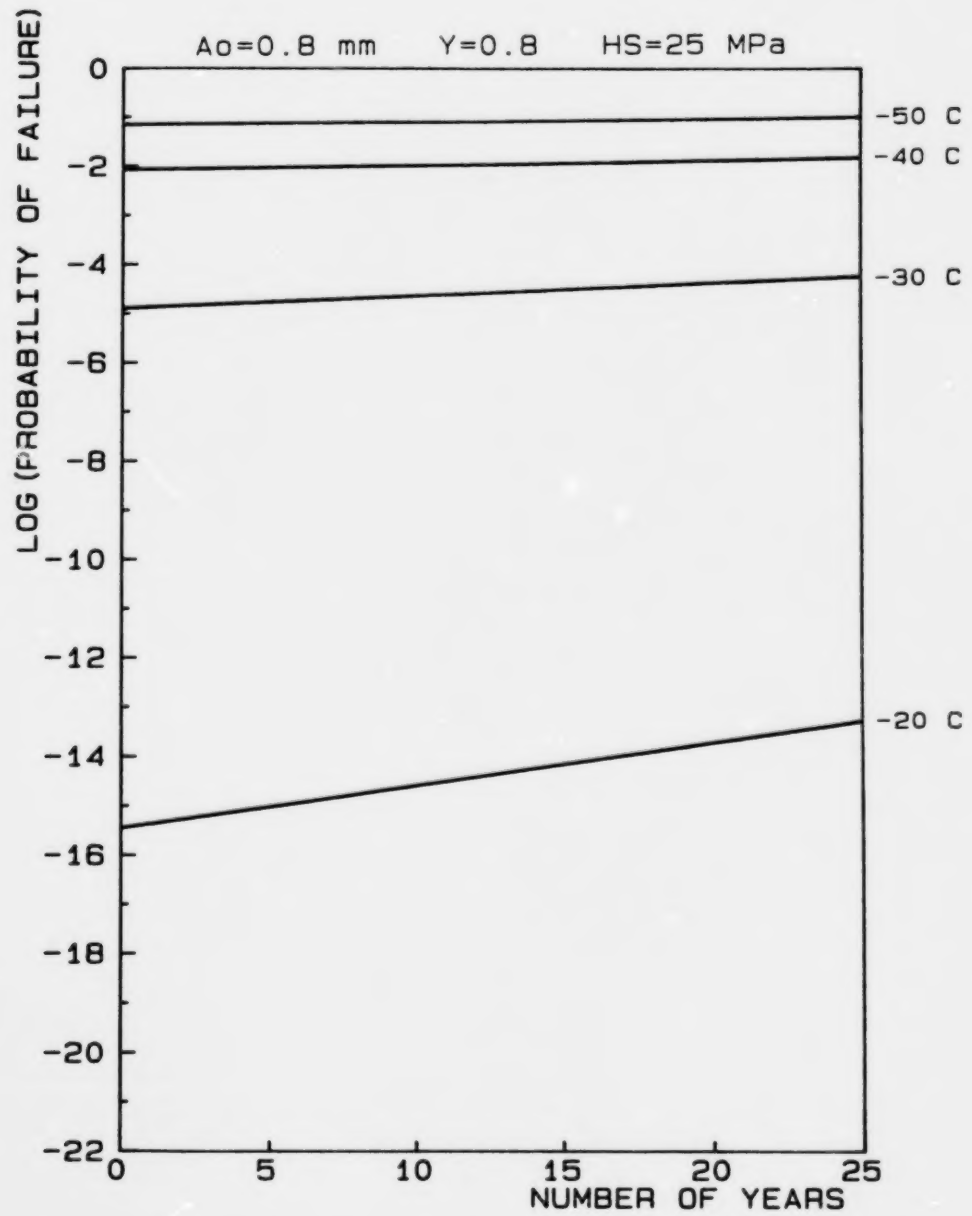


Figure C-18

- C15 -

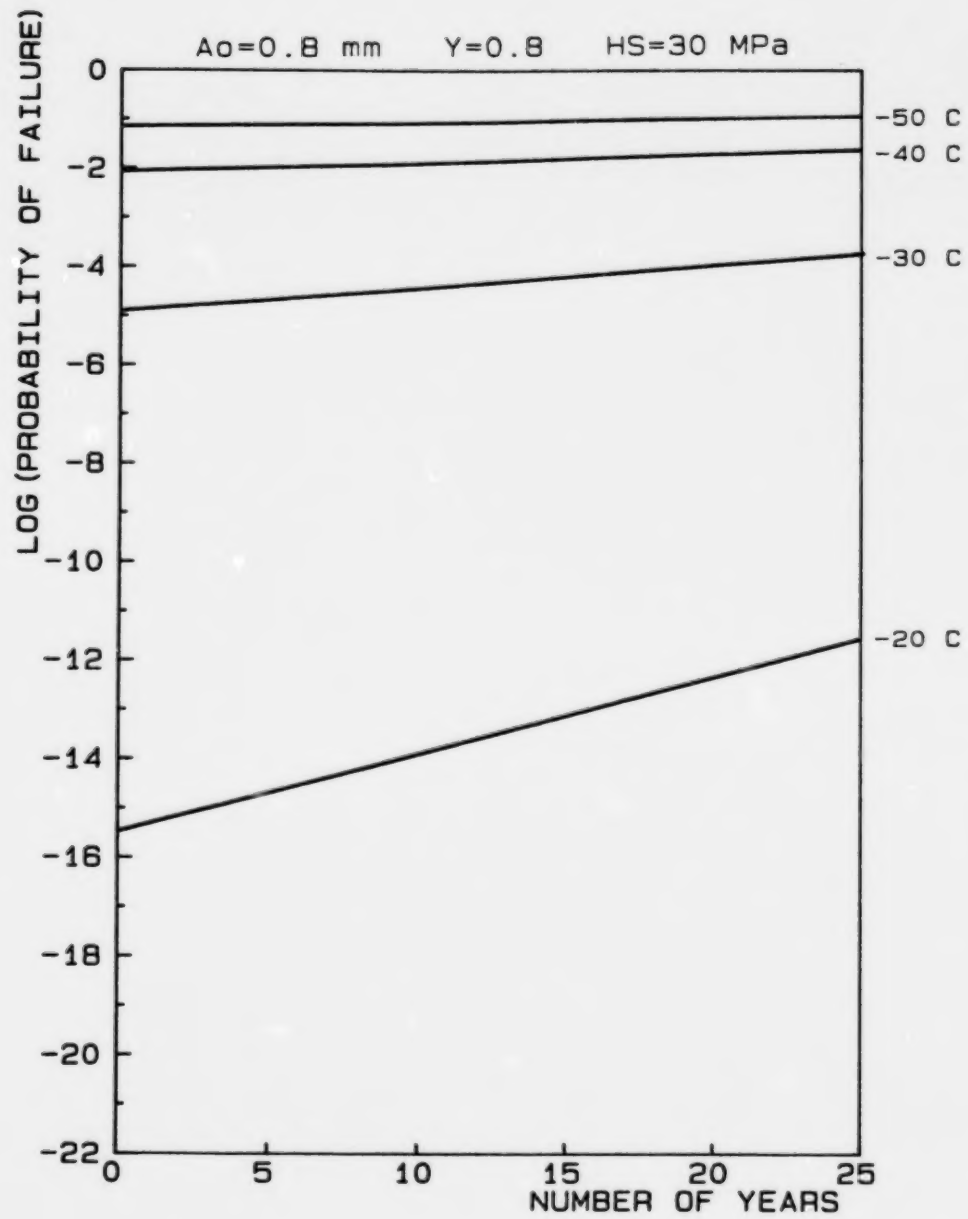


Figure C-19

- C16 -

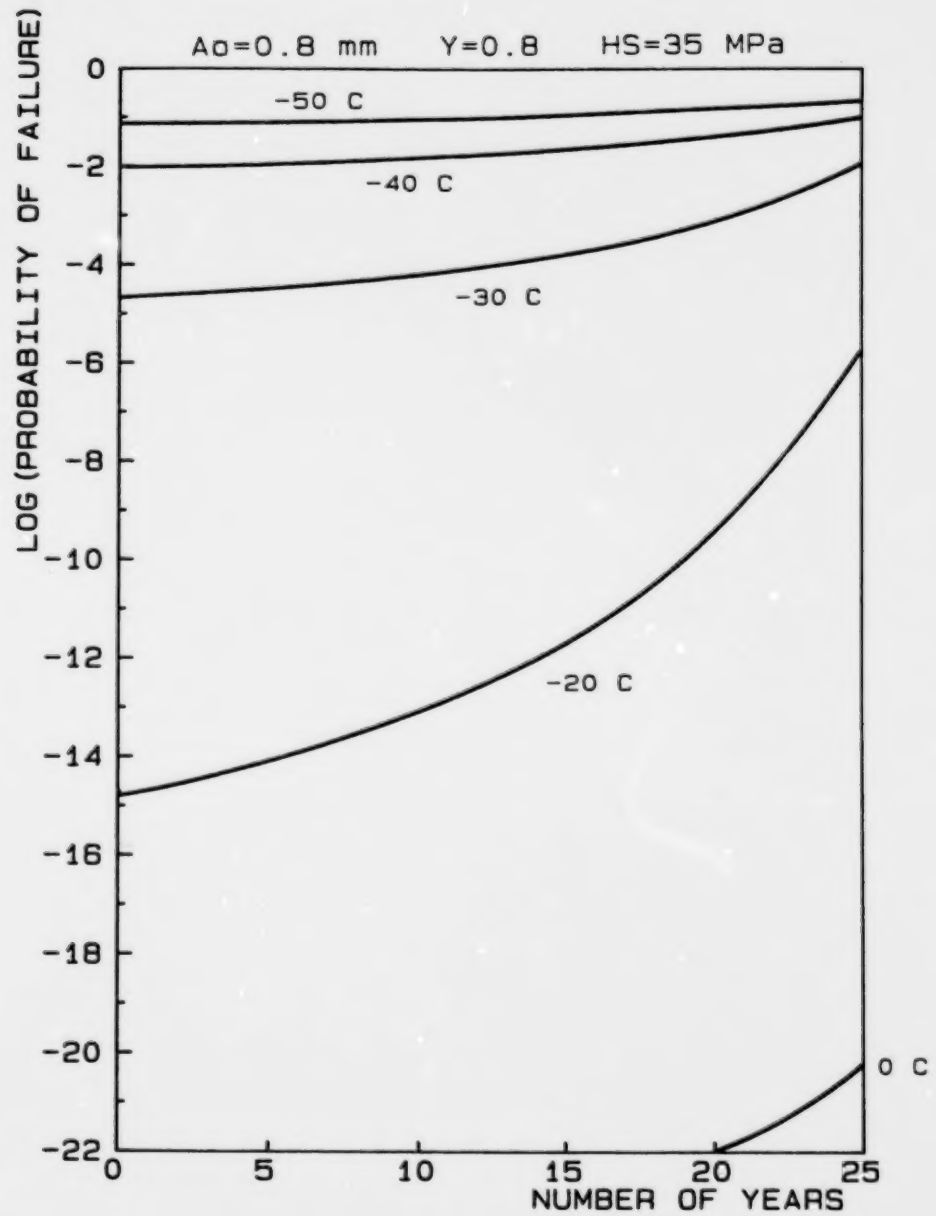


Figure C-20

- C17 -

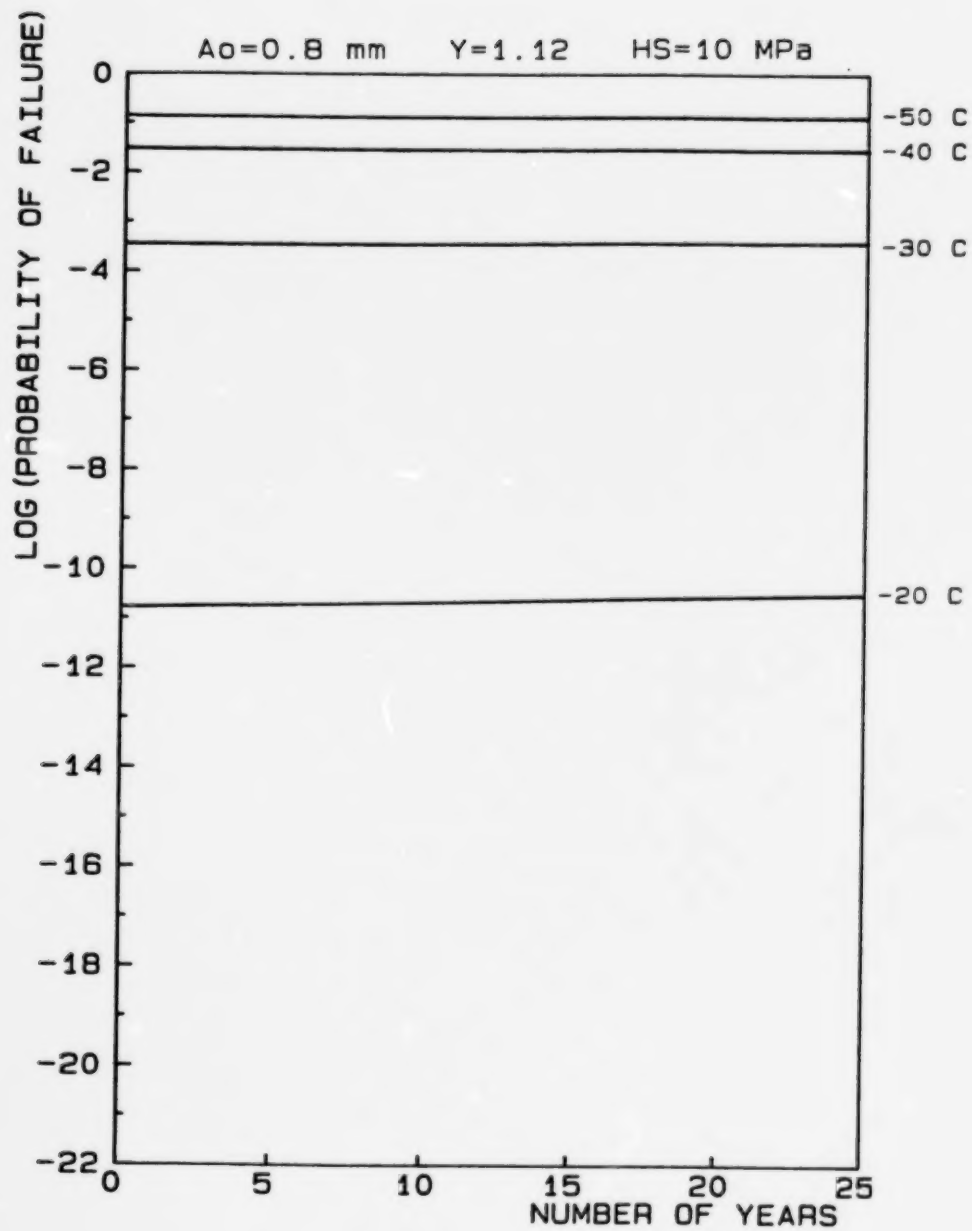


Figure C-21

- C18 -

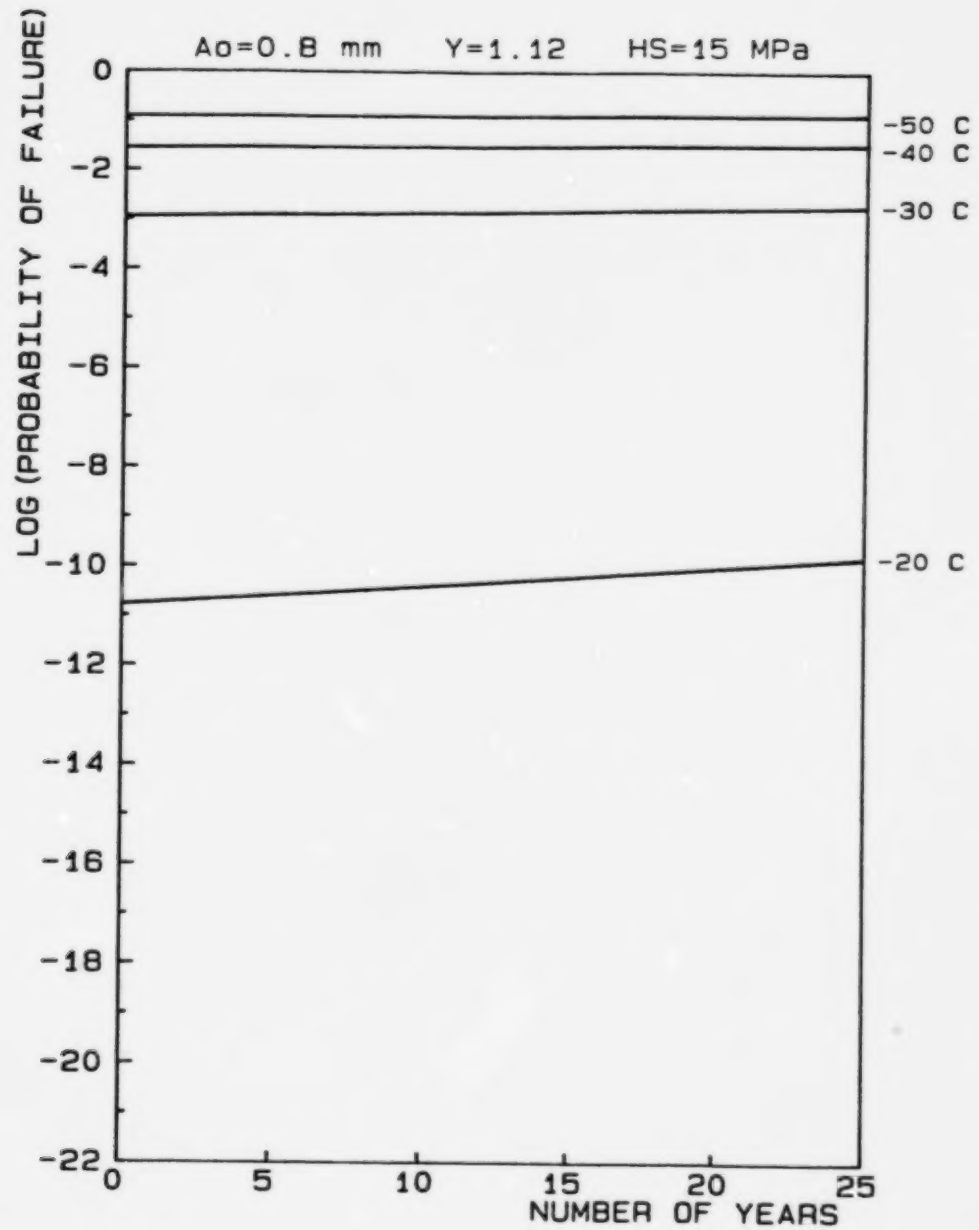


Figure C-22

- C19 -

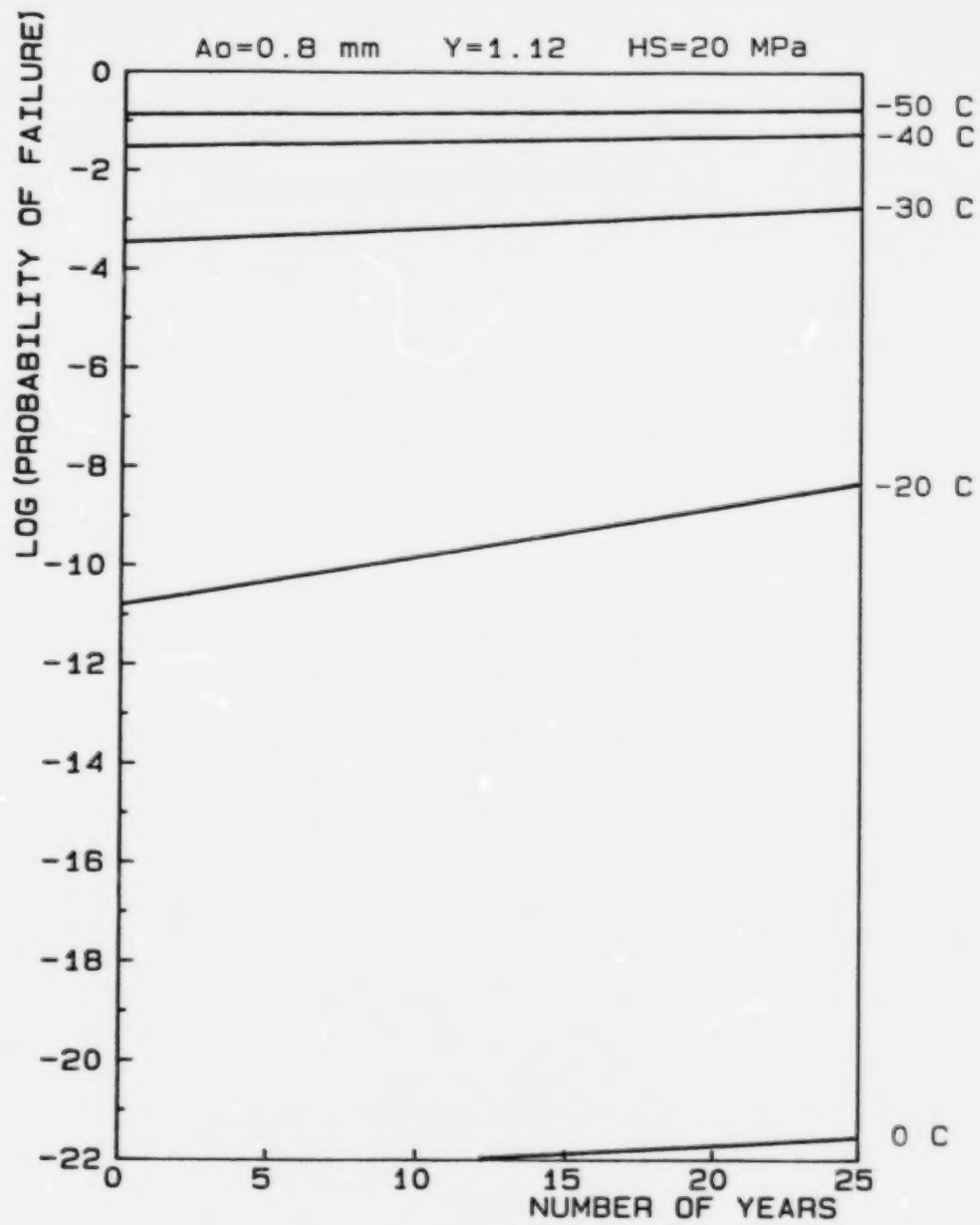


Figure C-23

- C20 -

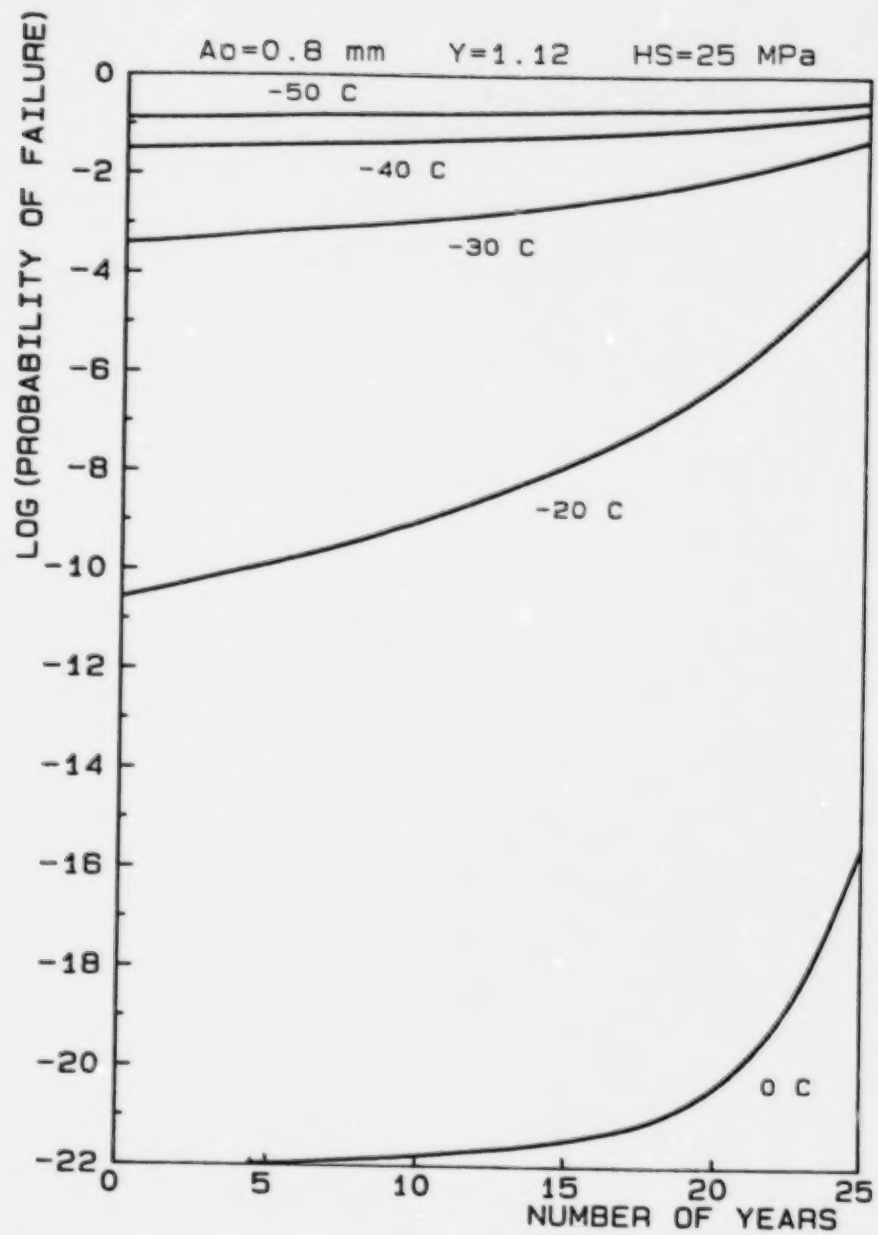


Figure C-24

- C21 -

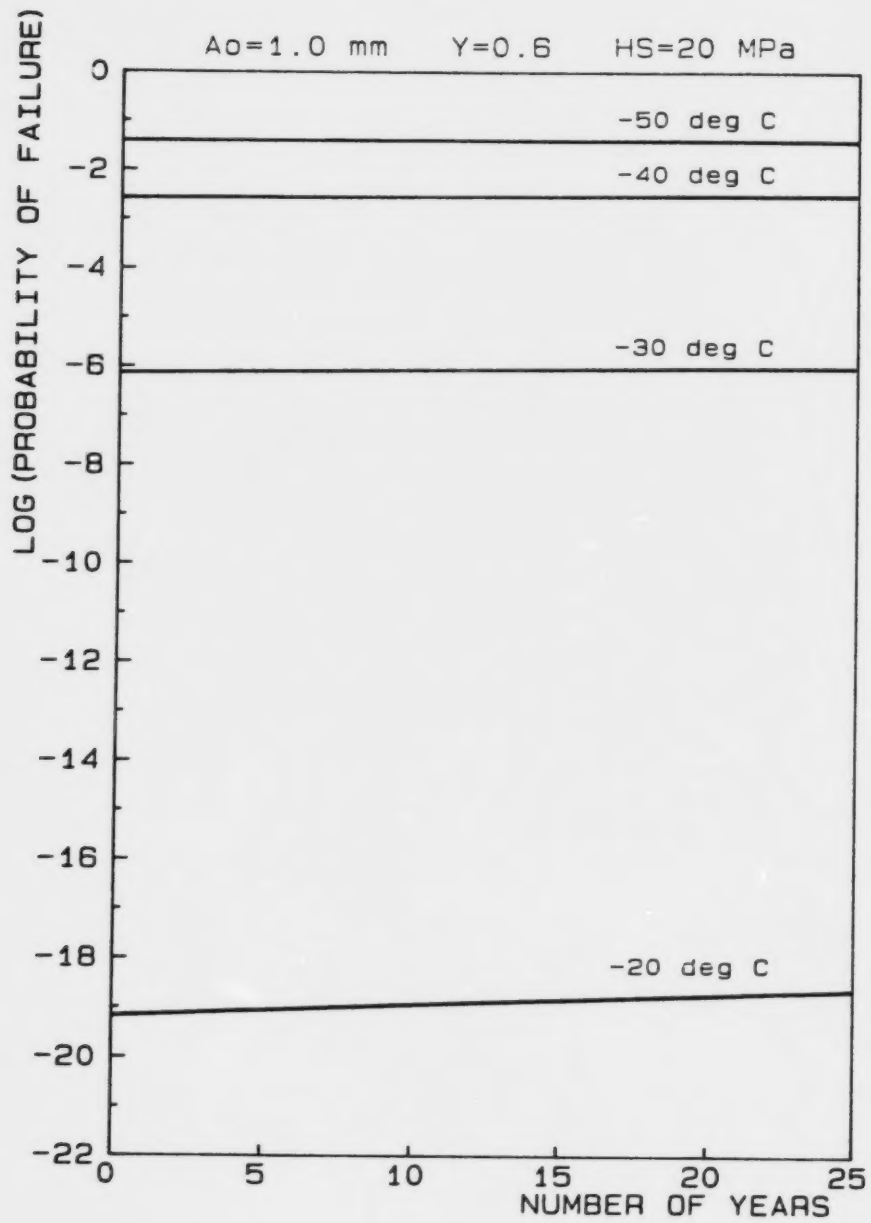


Figure C-25

- C22 -

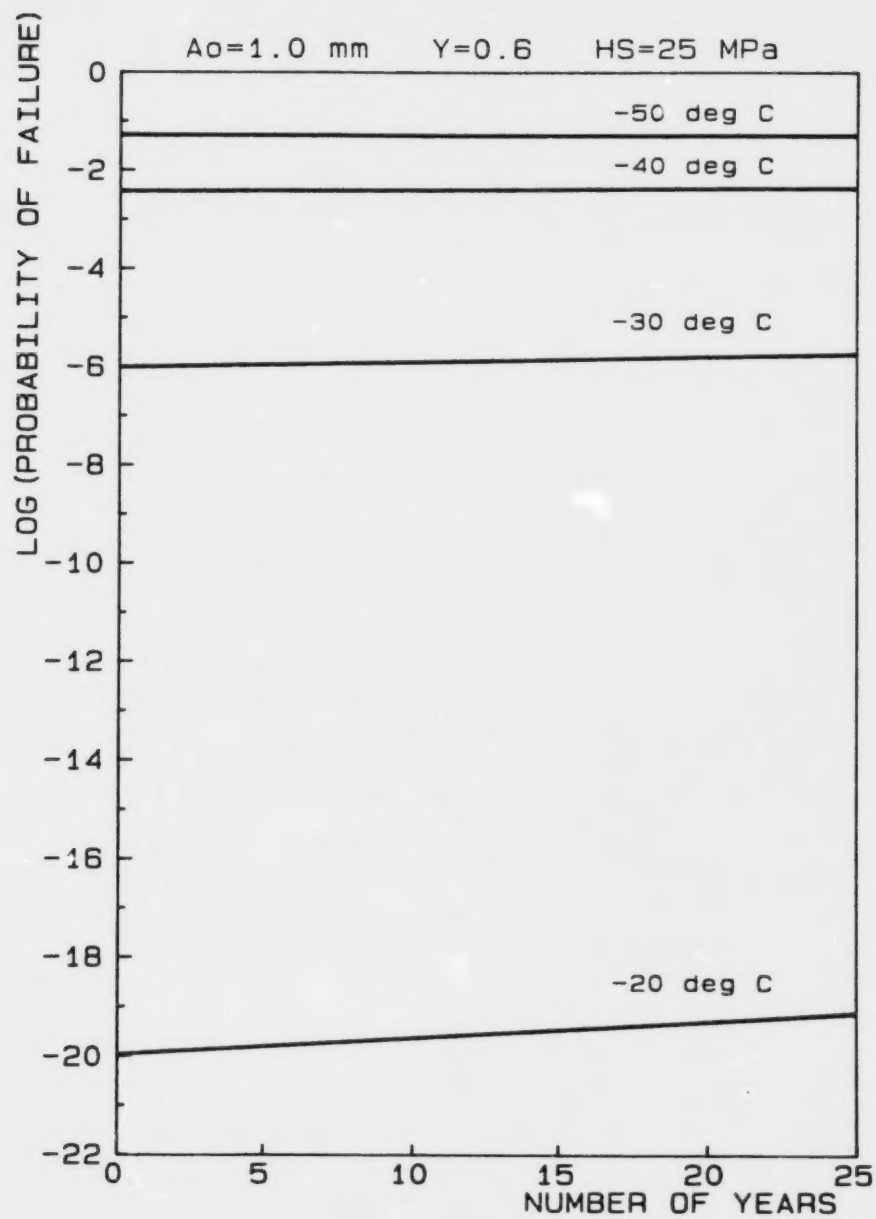


Figure C-26

- C23 -

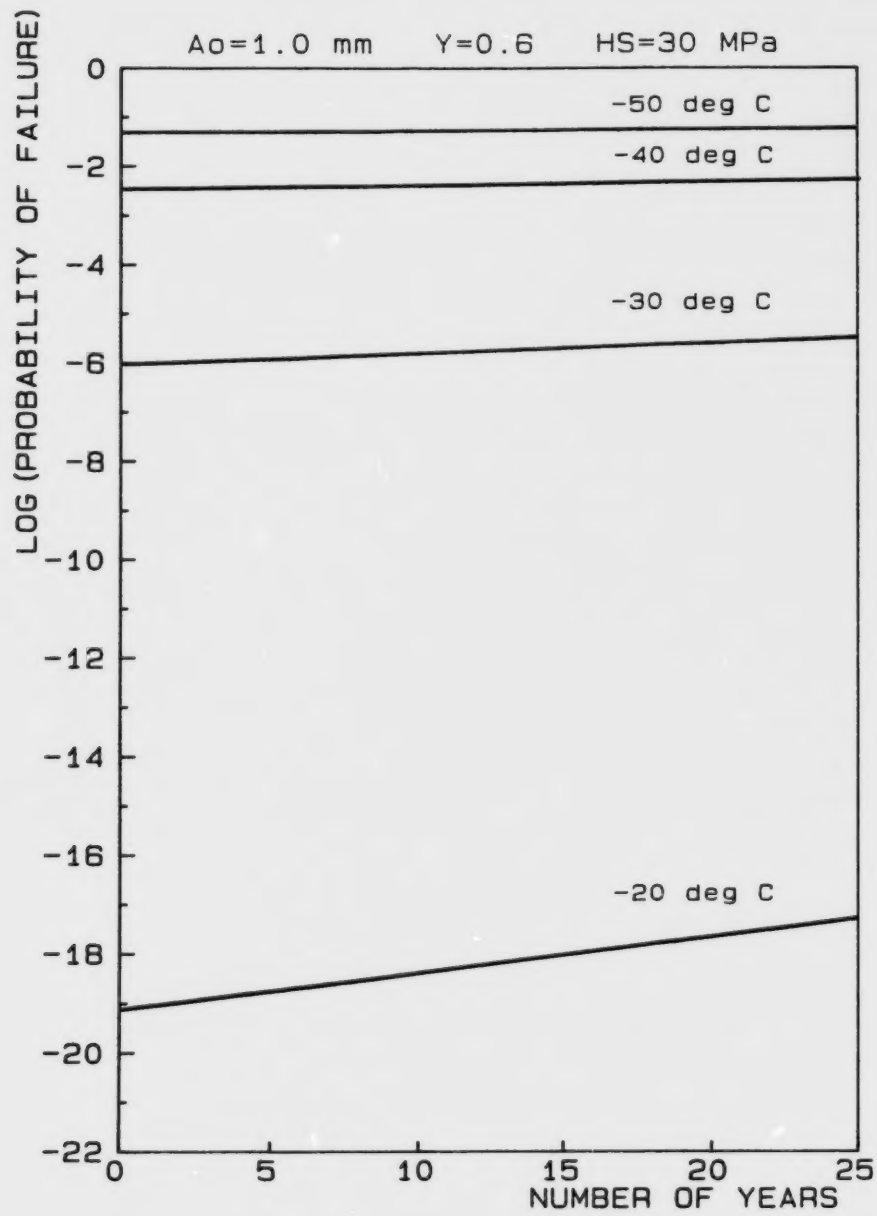


Figure C-27

- C24 -

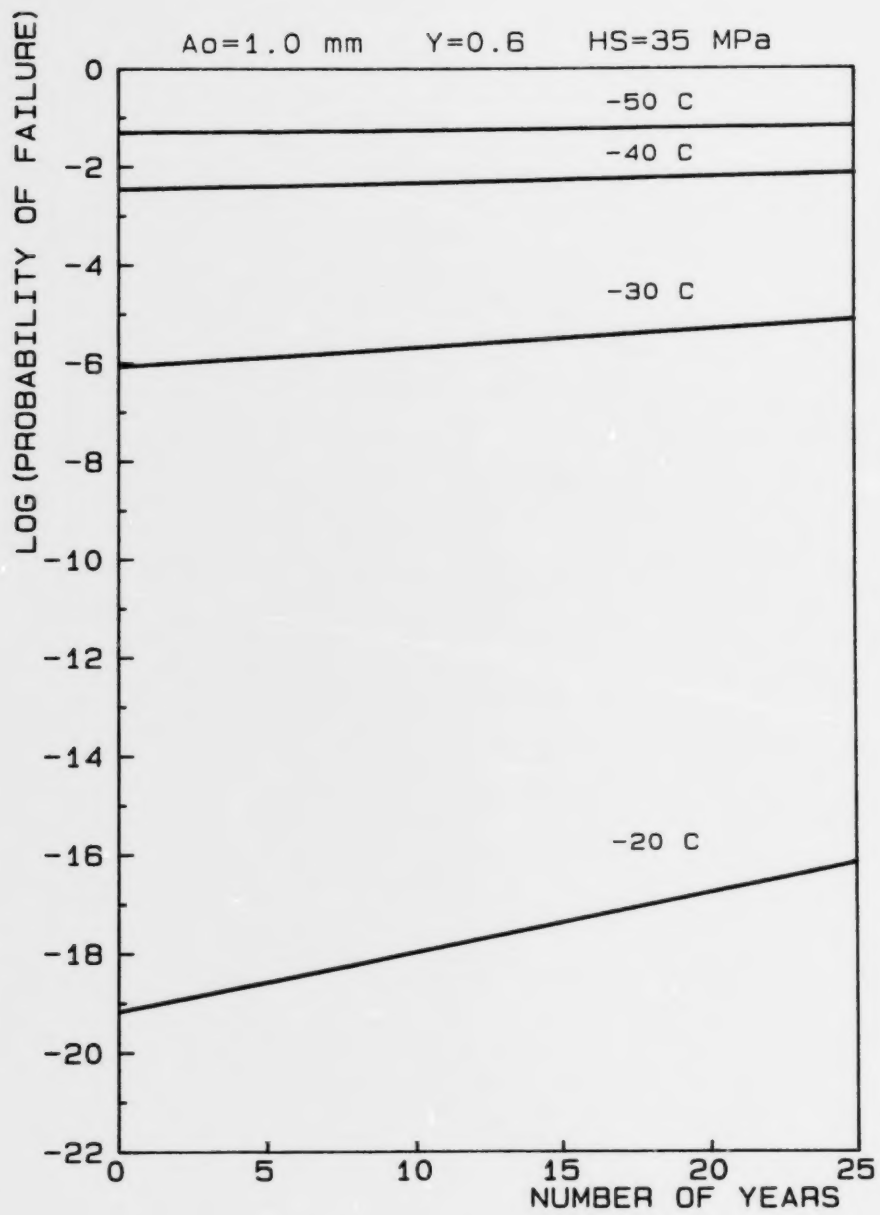


Figure C-28

- C25 -

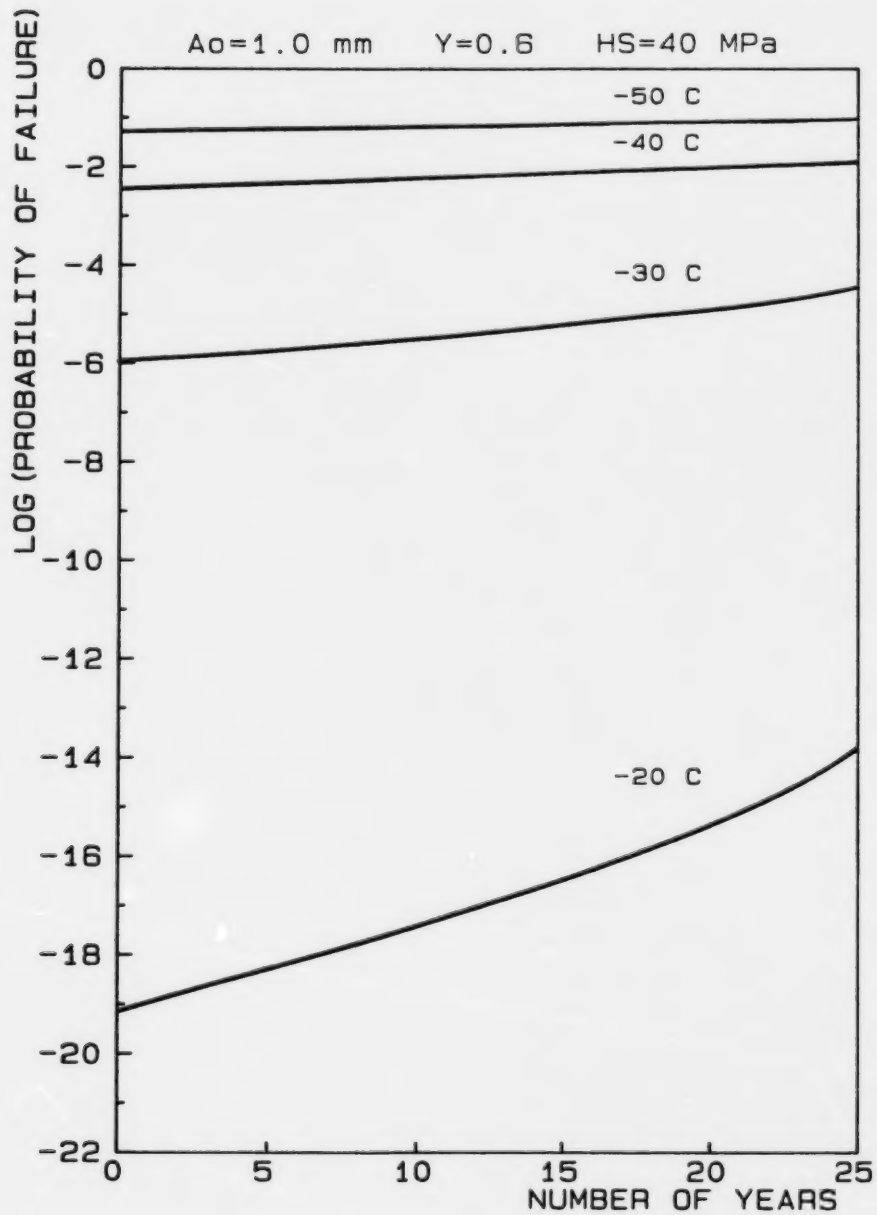


Figure C-29

- C26 -

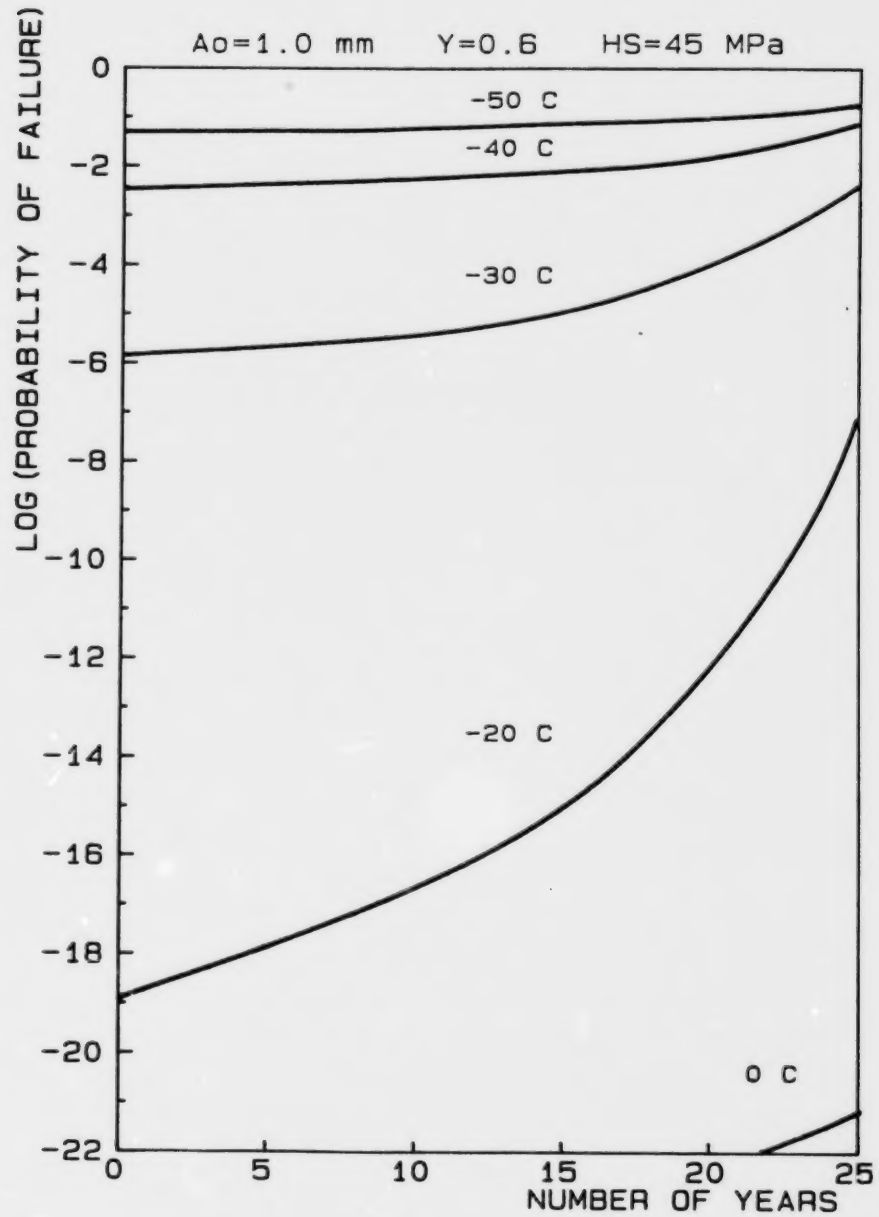


Figure C-30

- C27 -

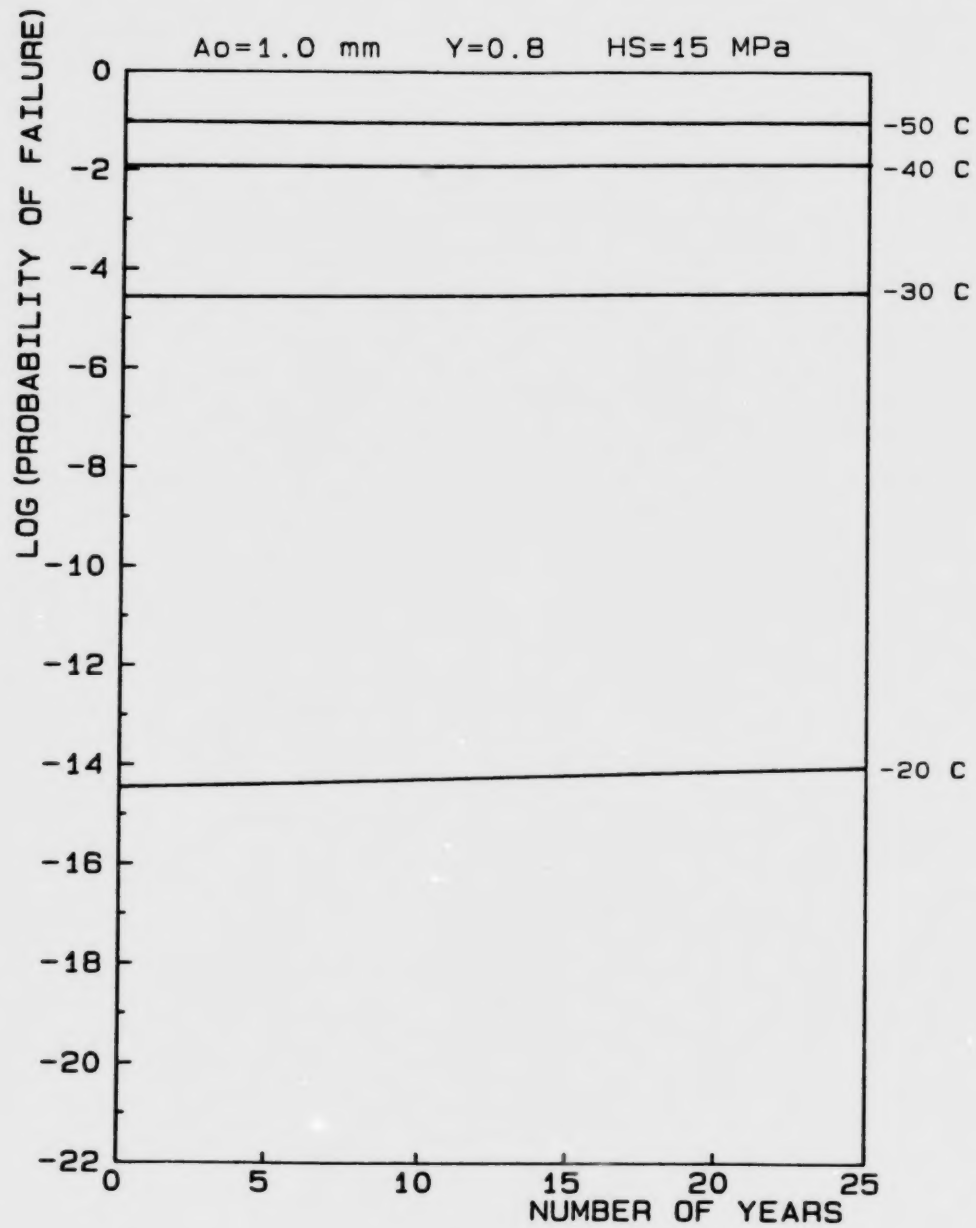


Figure C-31

- C28 -

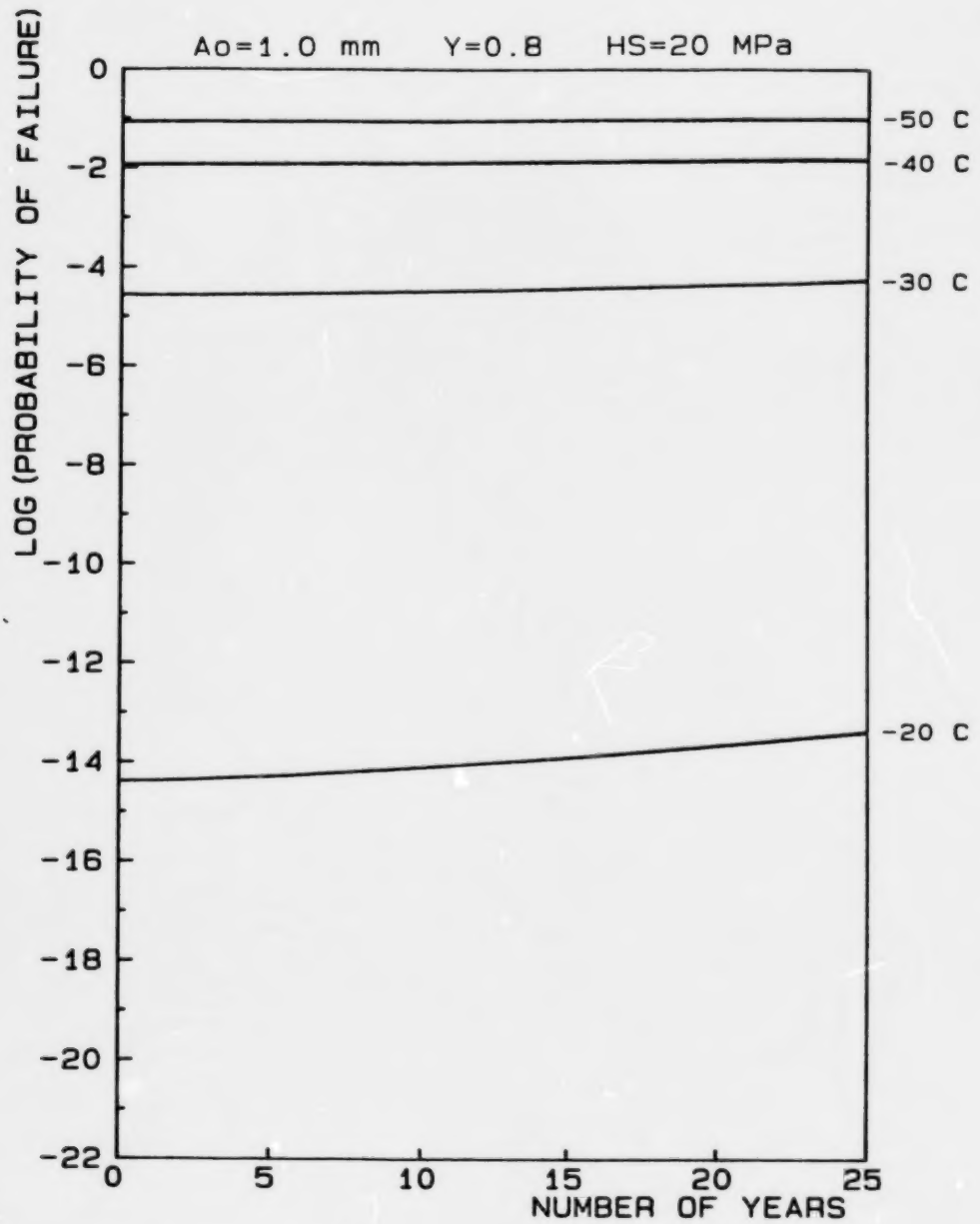


Figure C-32

- C29 -

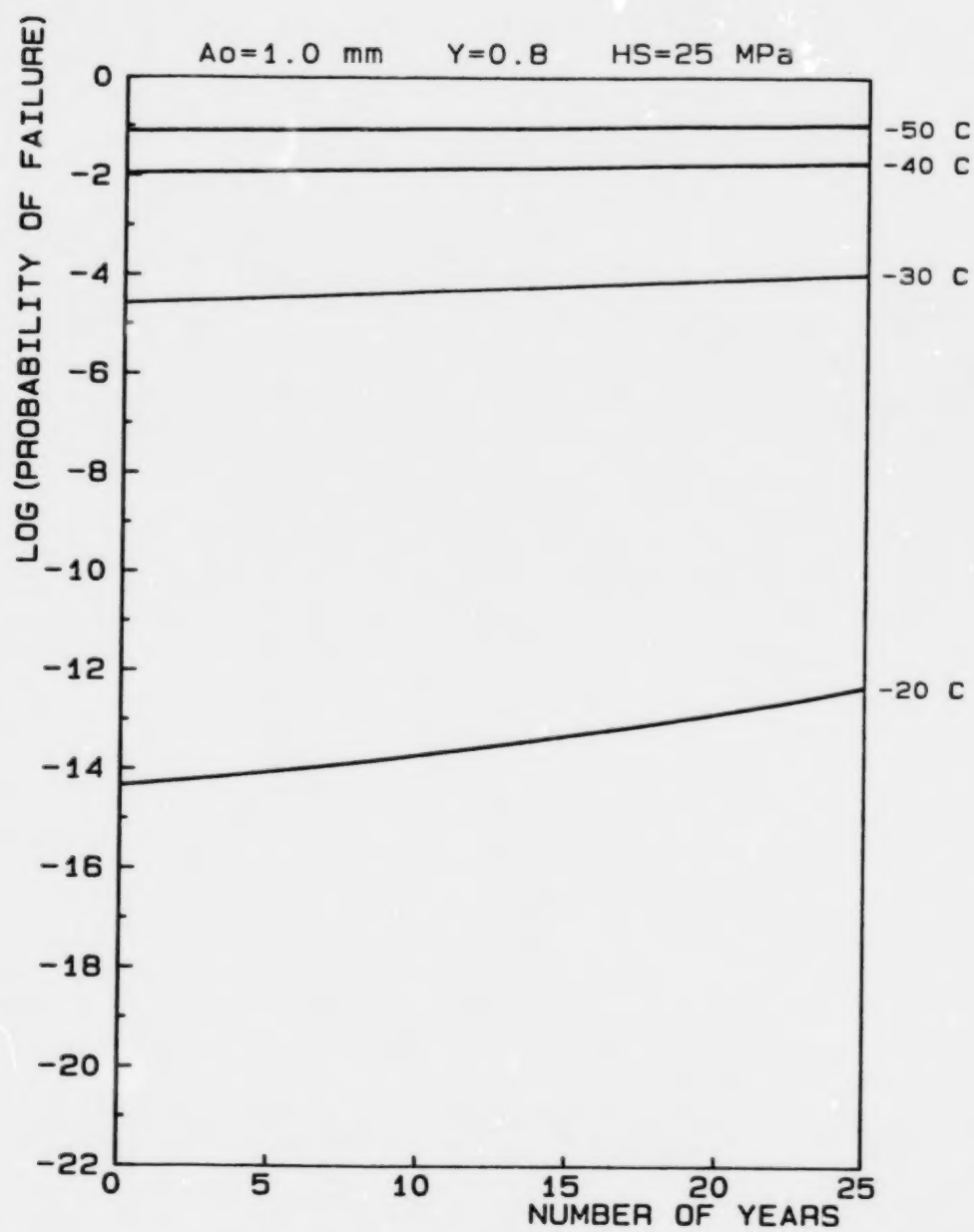


Figure C-33

- C30 -

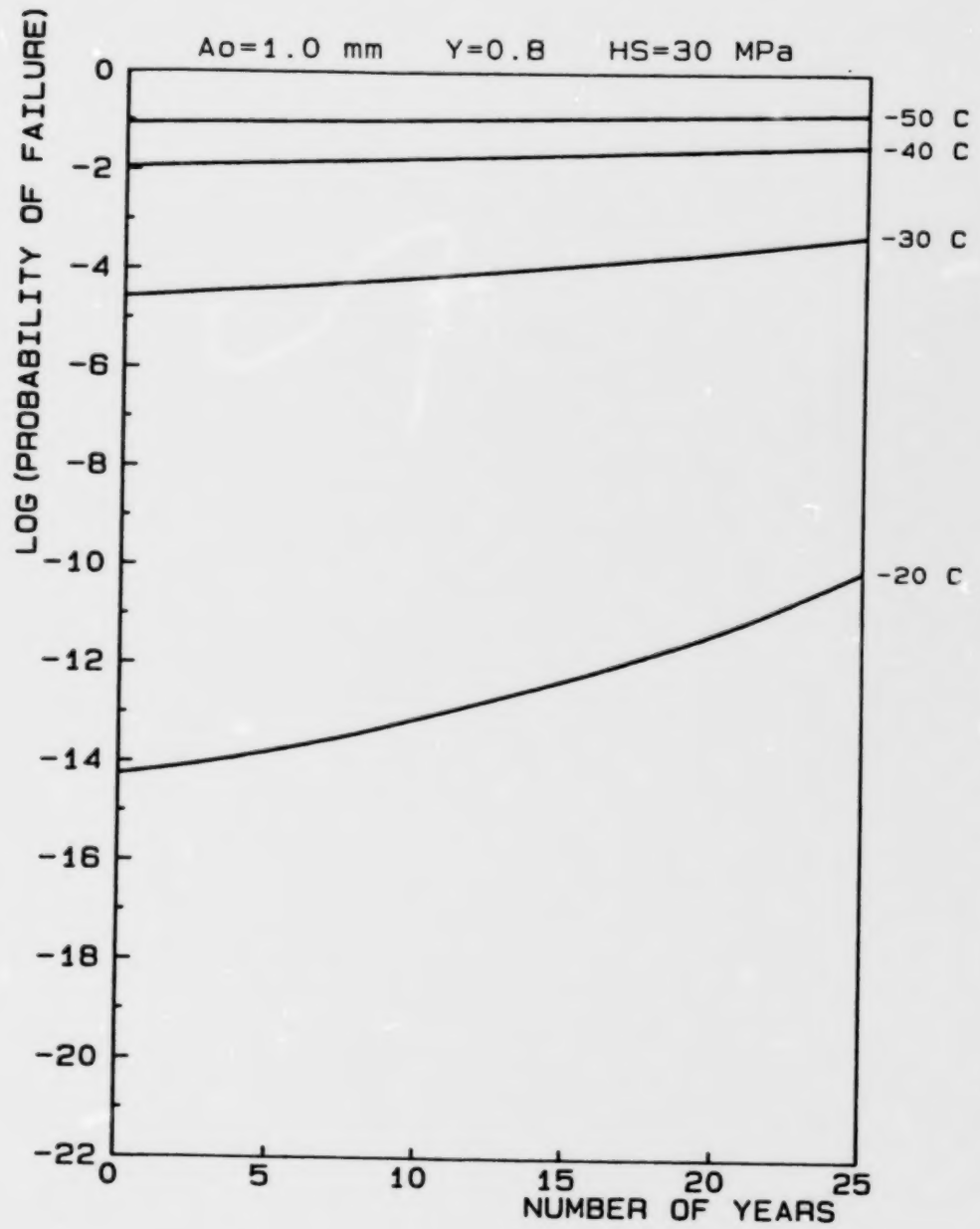


Figure C-34

- C31 -

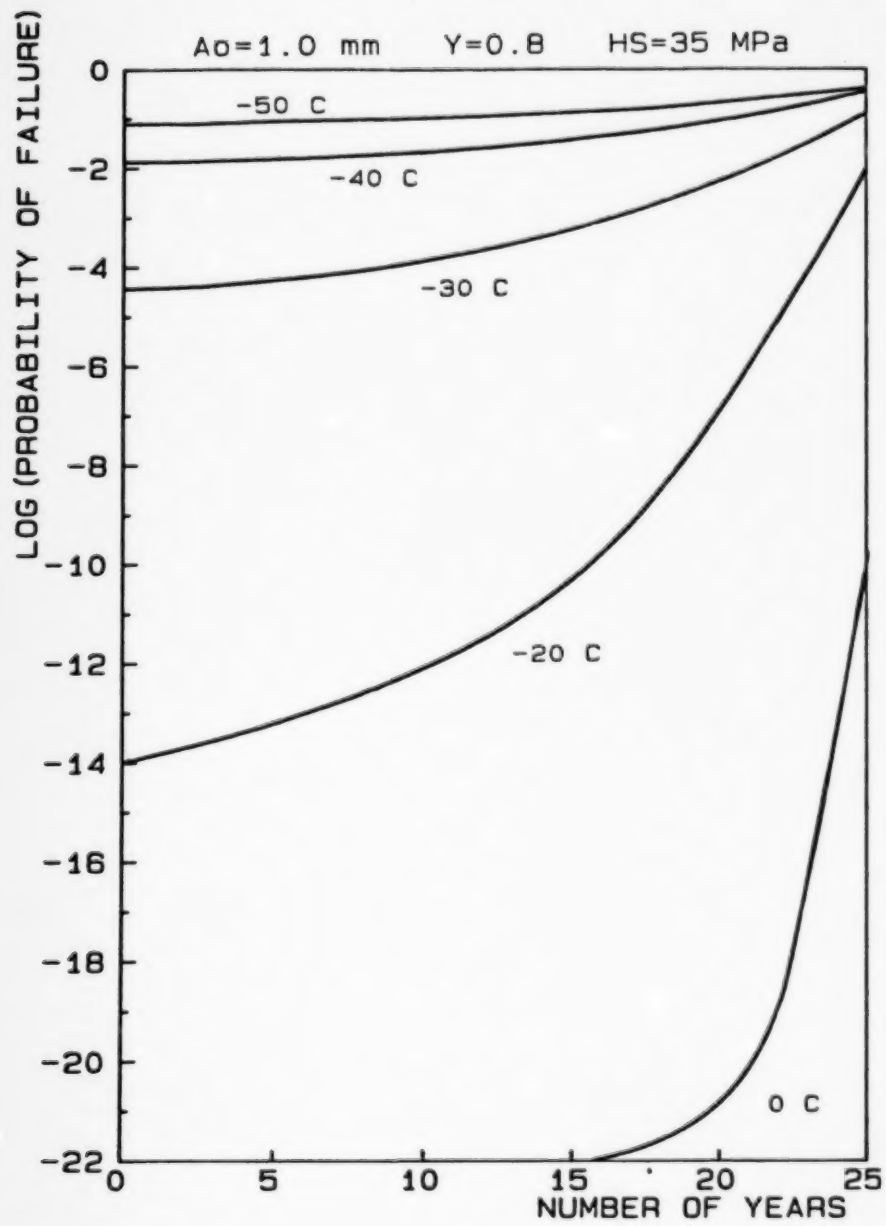


Figure C-35

- C32 -

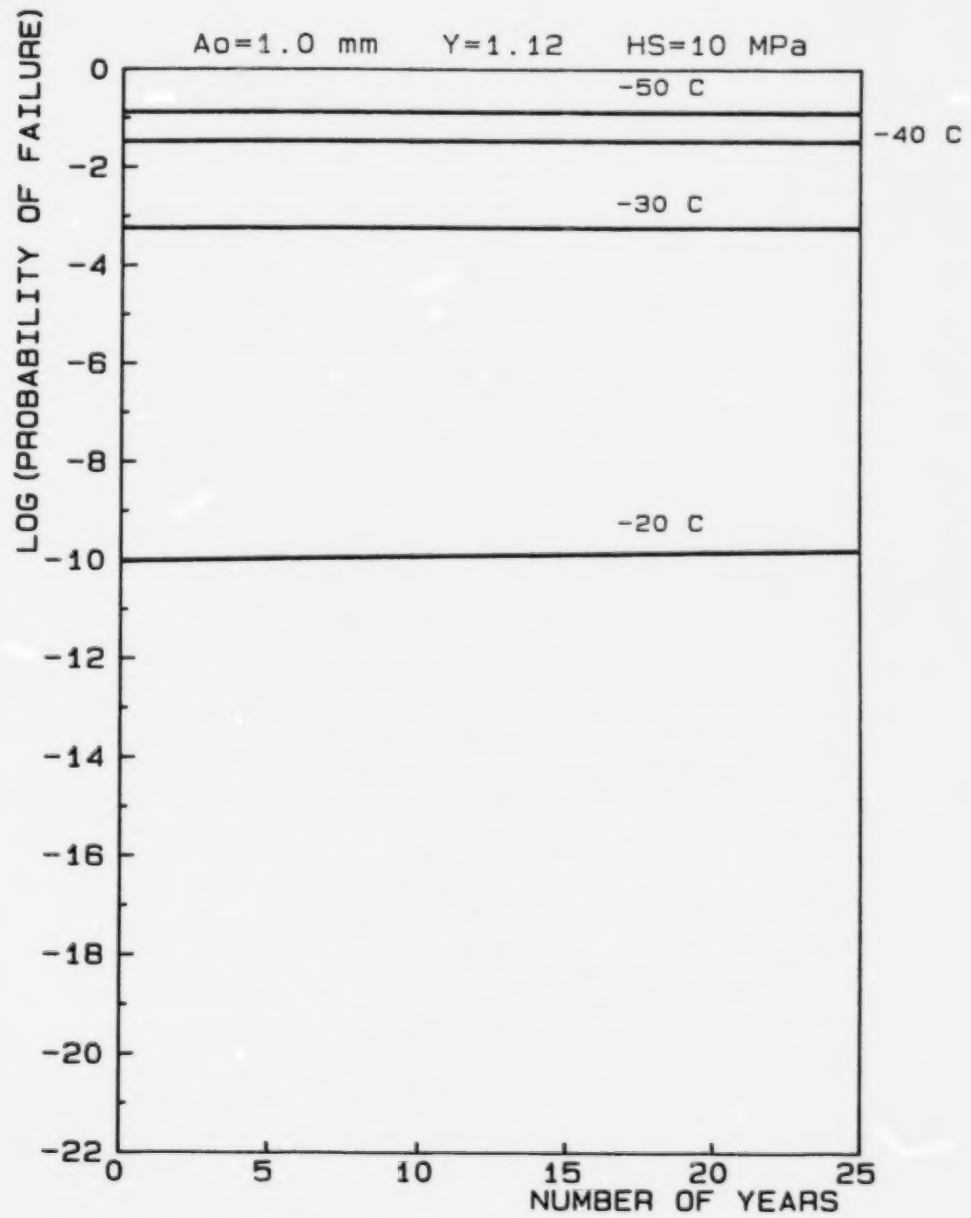


Figure C-36

- C33 -

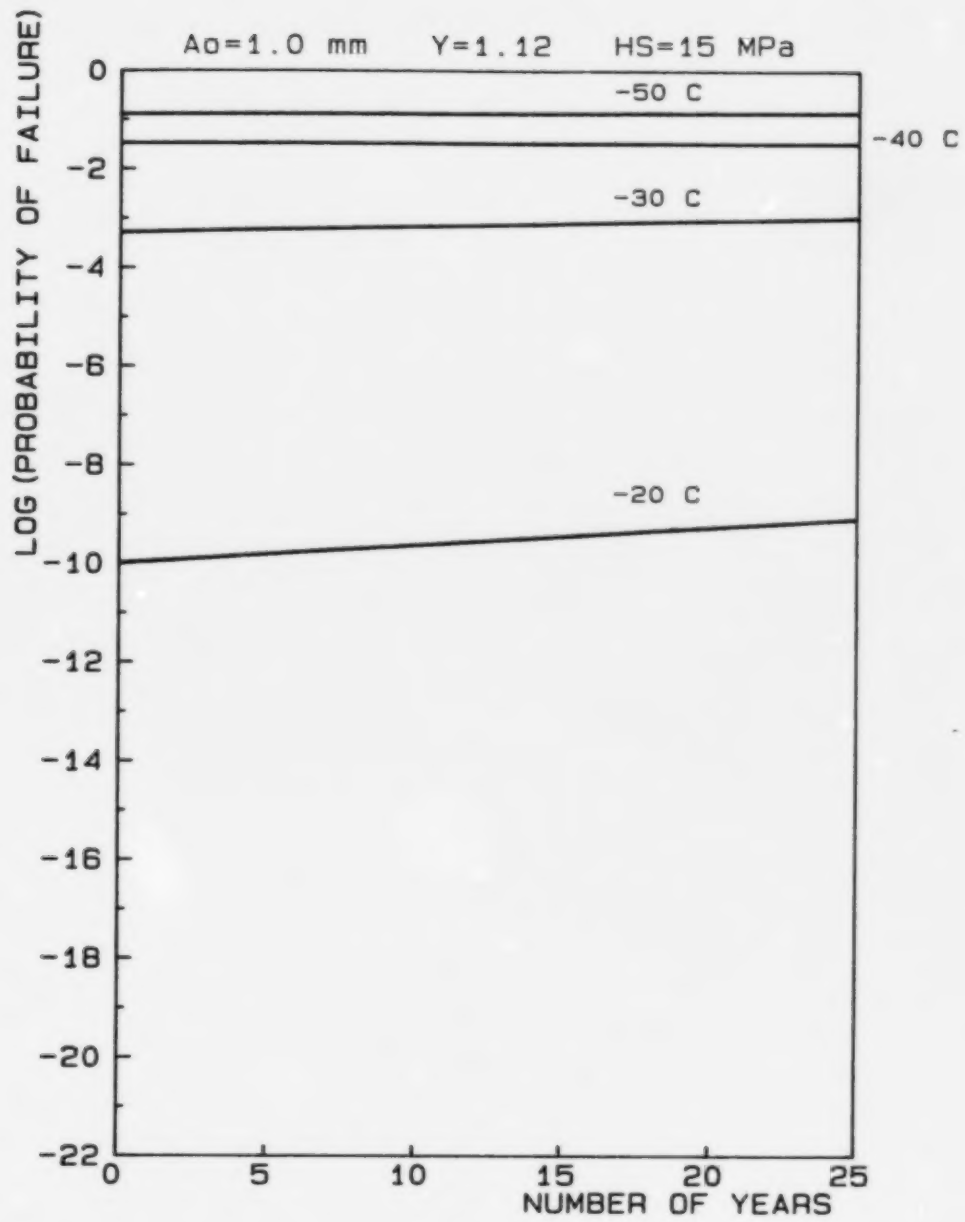


Figure C-37

- C34 -

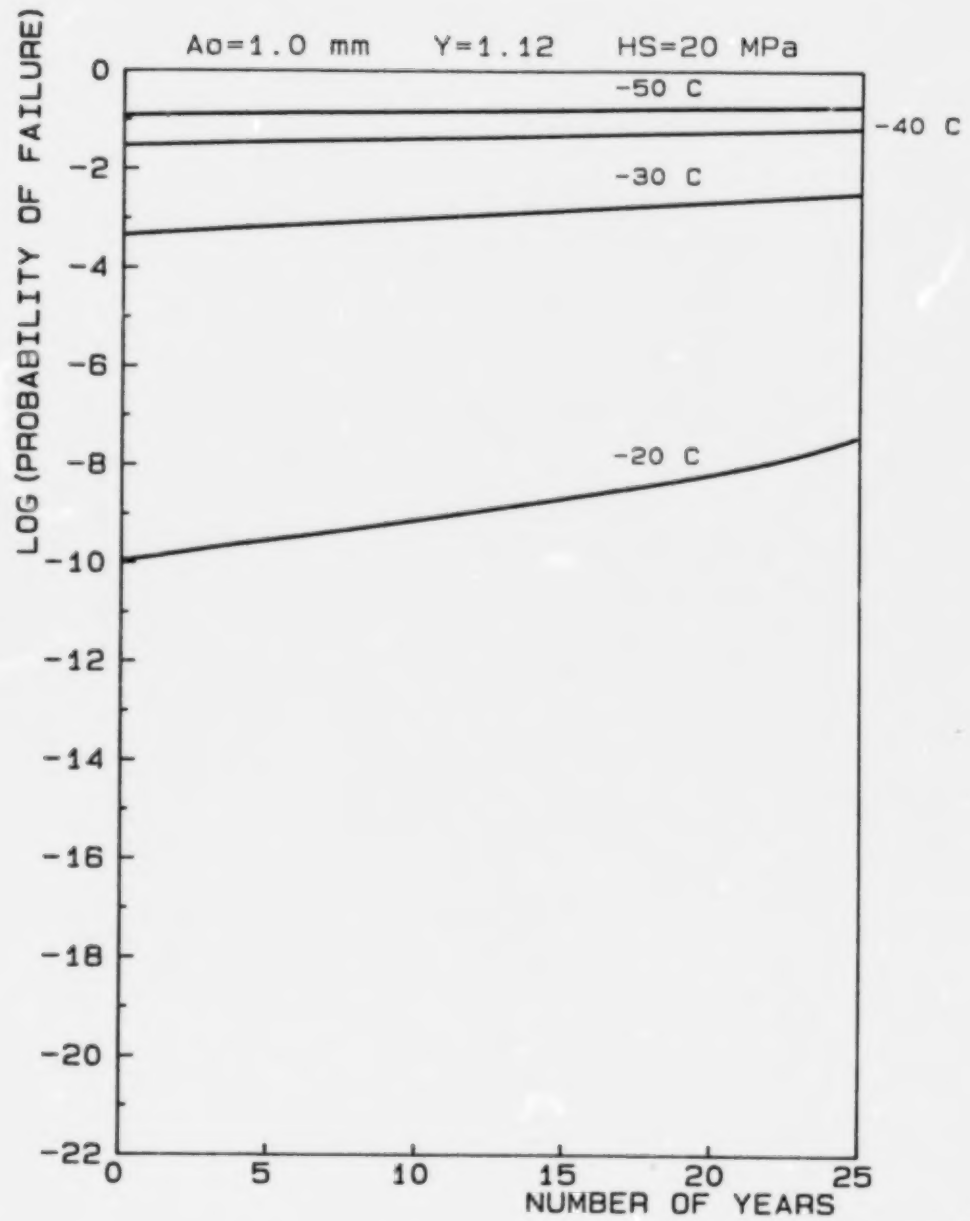


Figure C-38

- C35 -

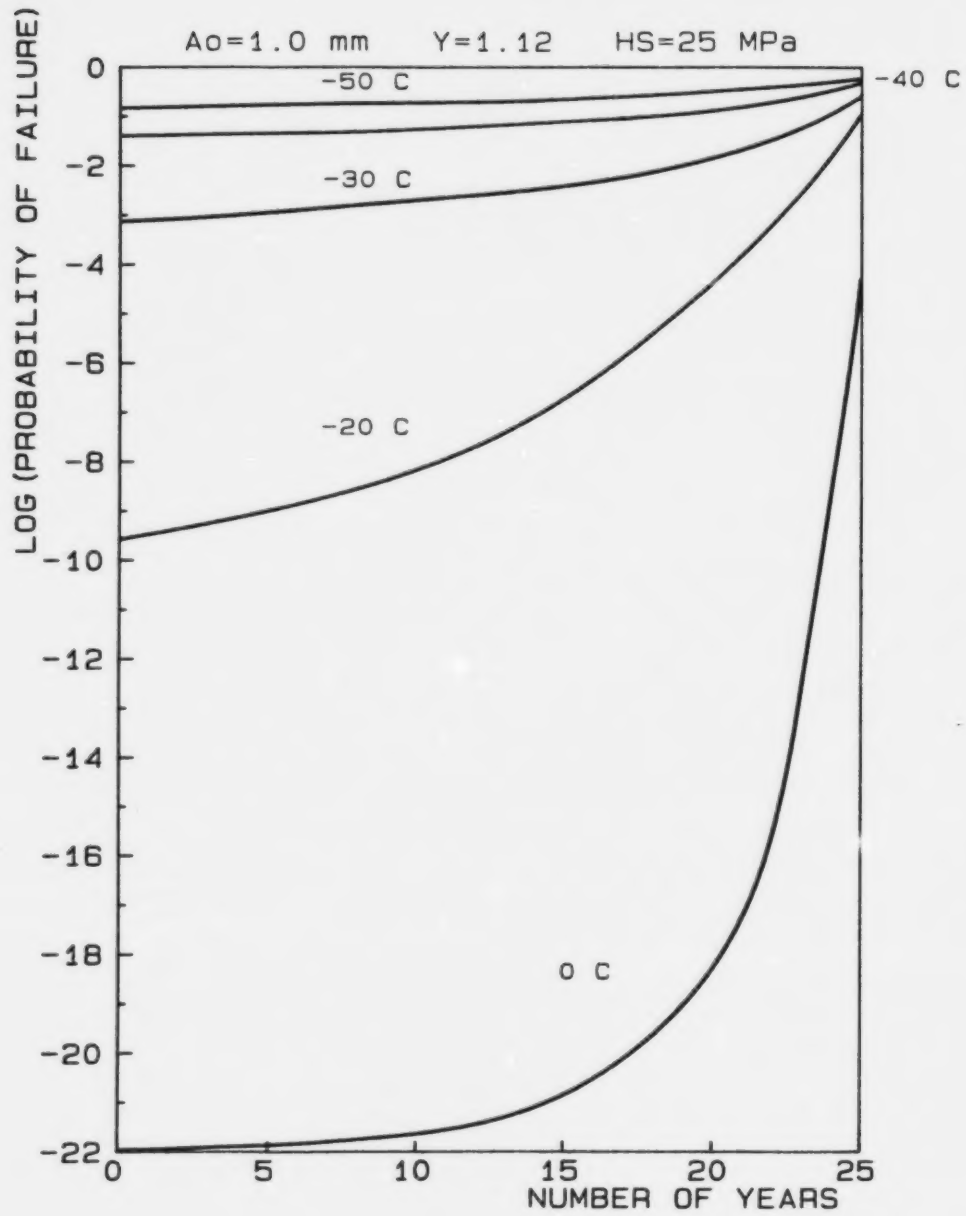


Figure C-39

- C36 -

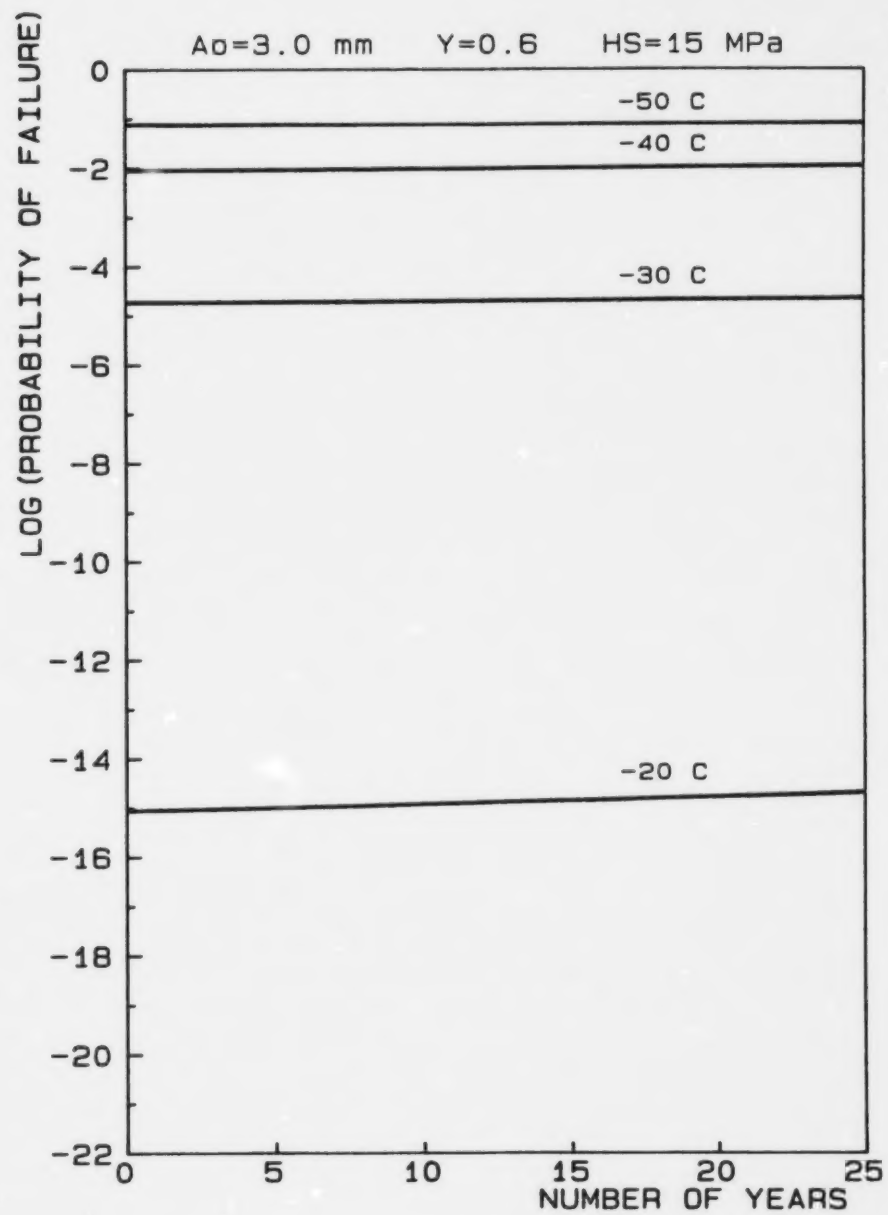


Figure C-40

- C37 -

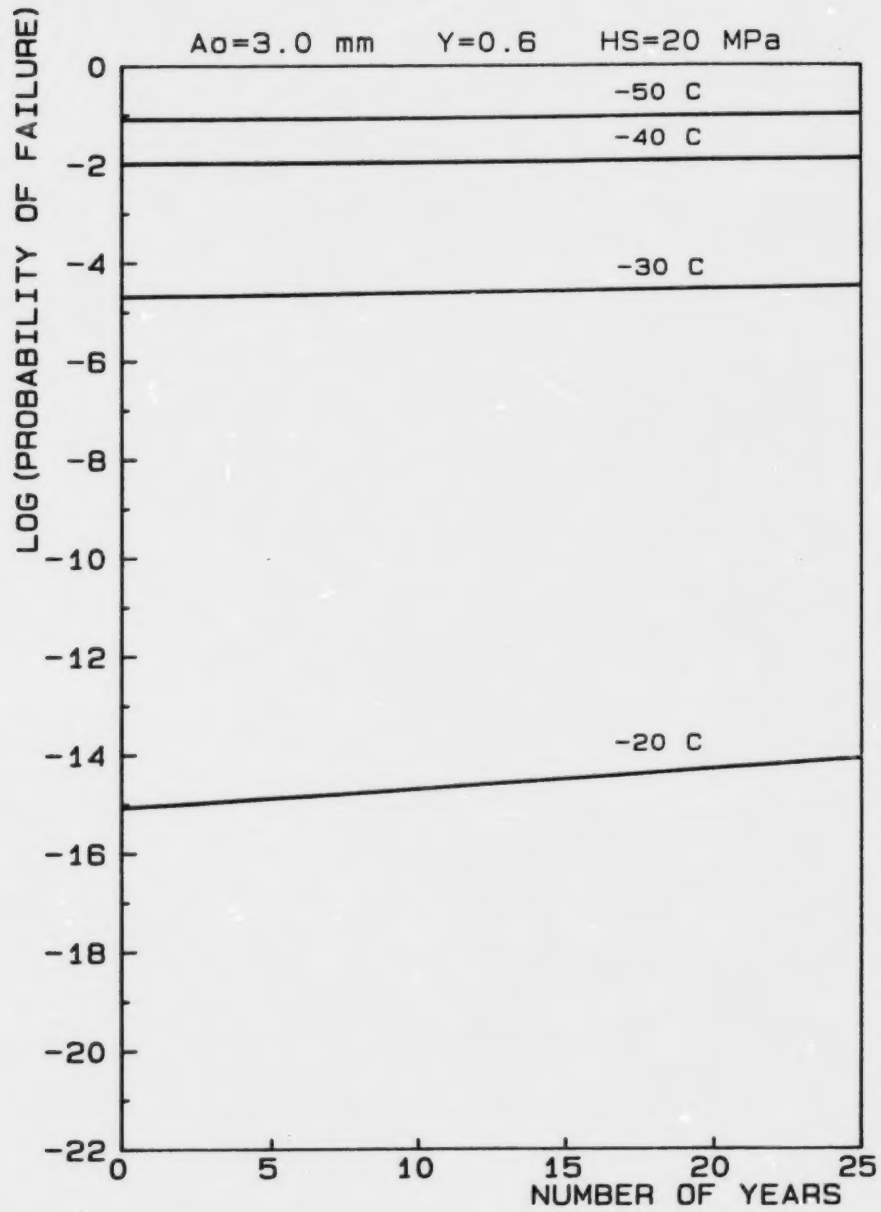


Figure C-41

- C38 -

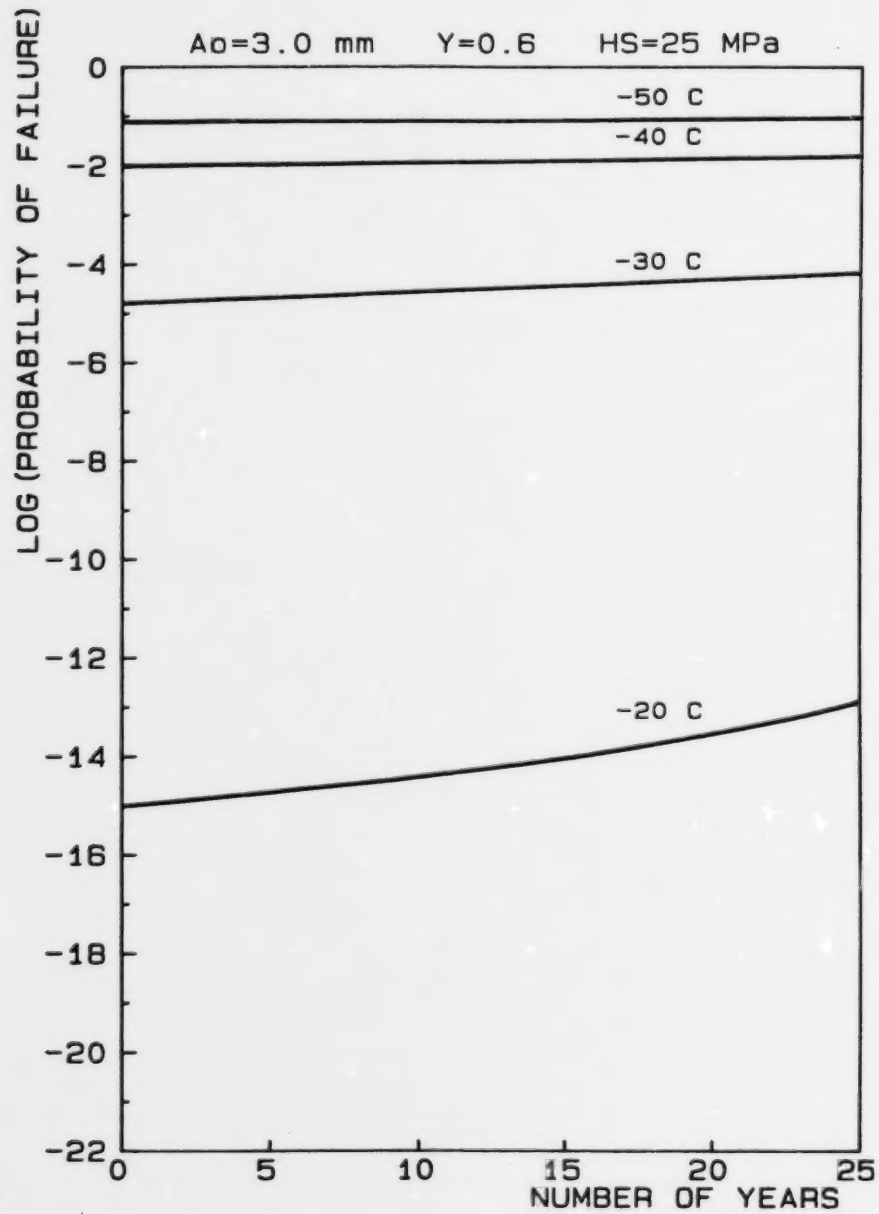


Figure C-42

- C39 -

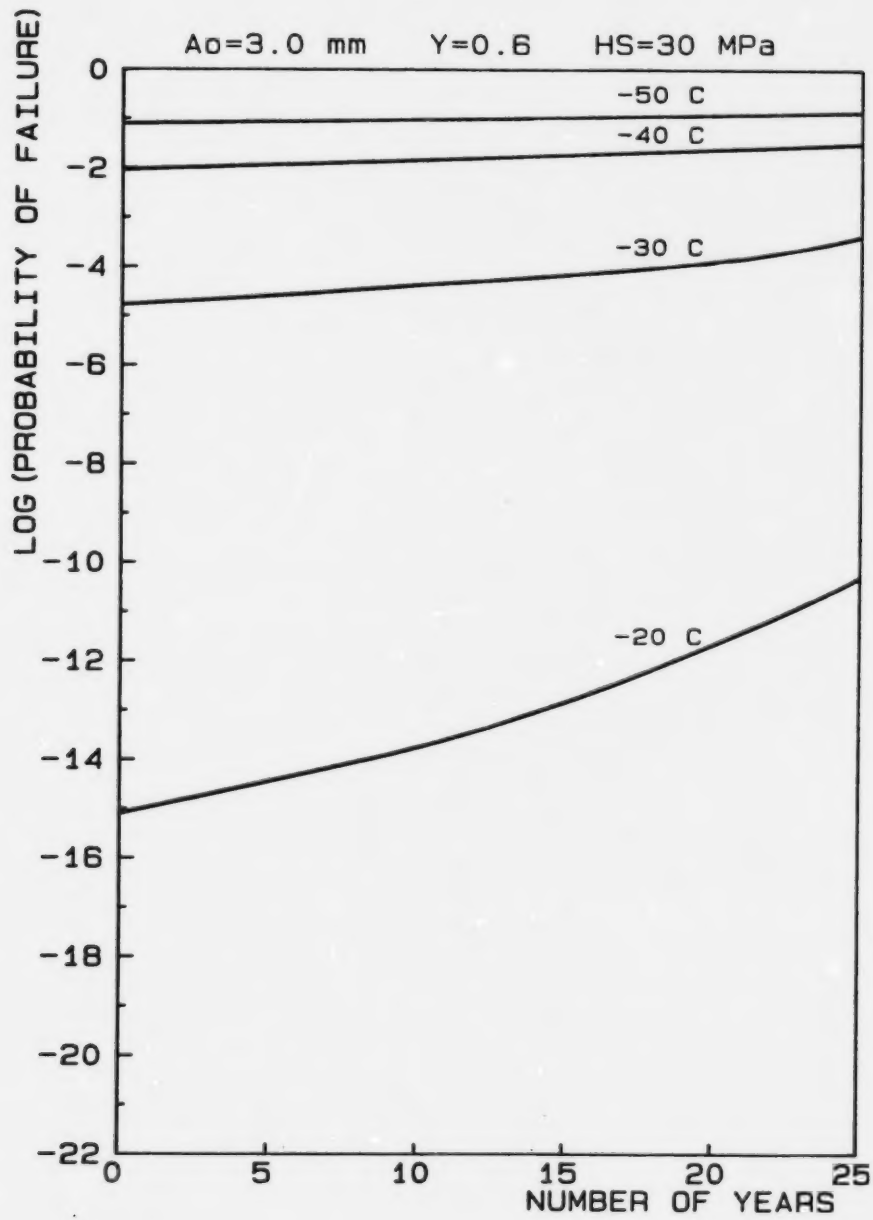


Figure C-43

- C40 -

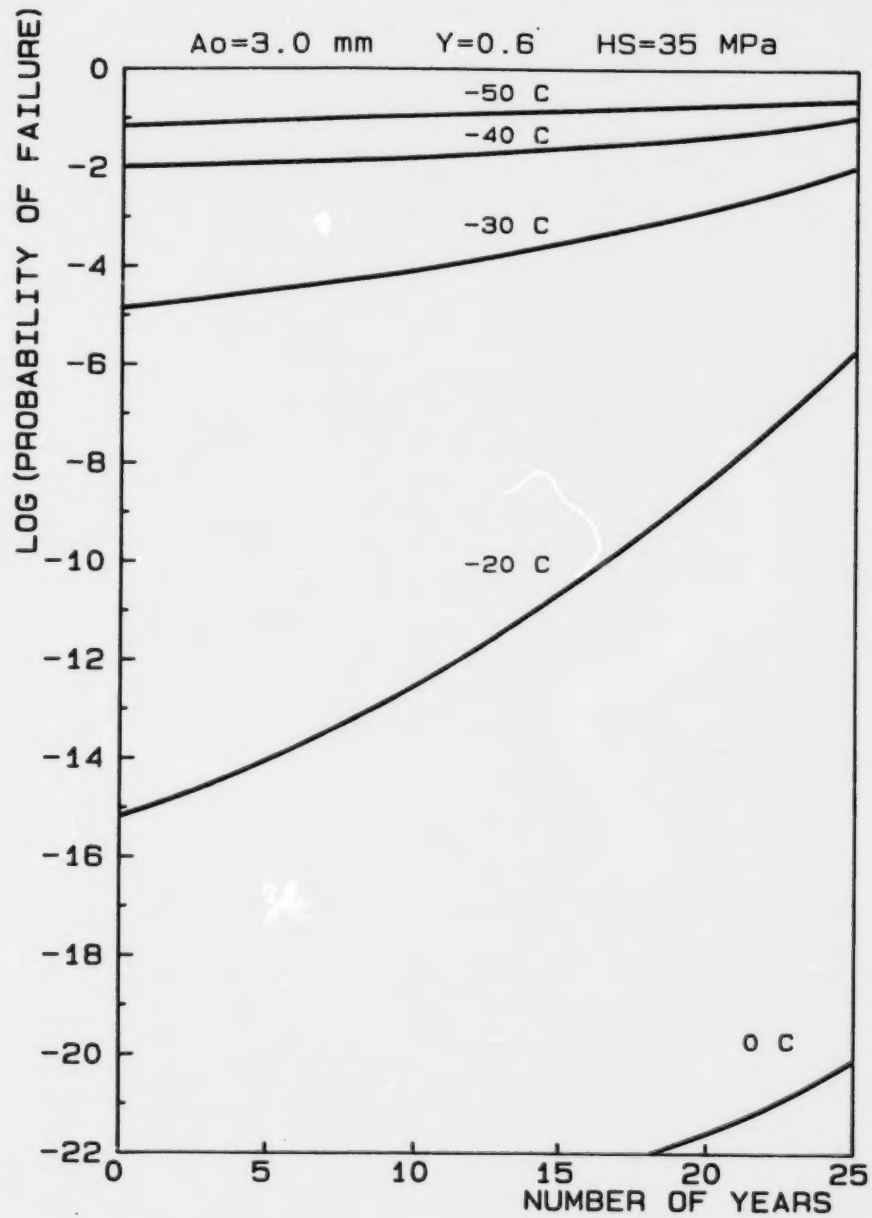


Figure C-44

- C41 -

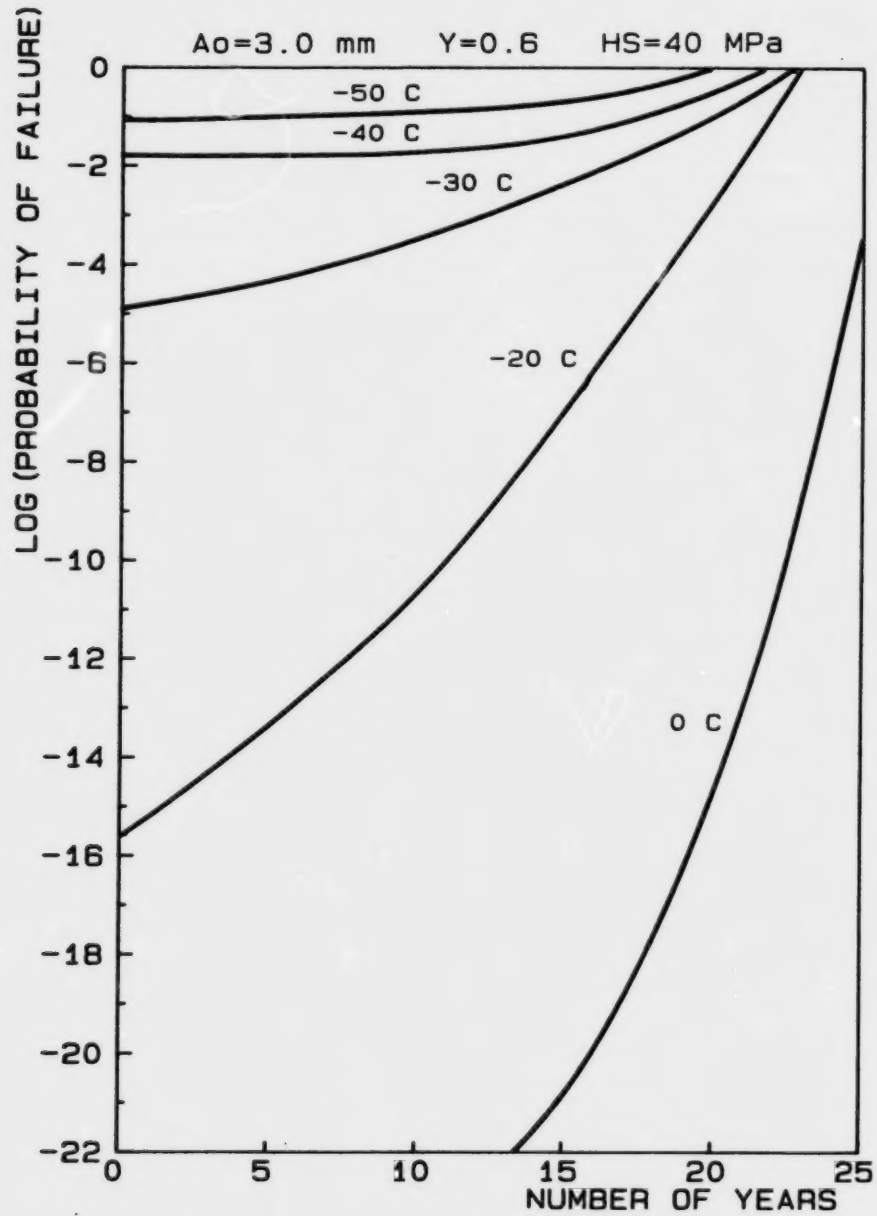


Figure C-45

- C42 -

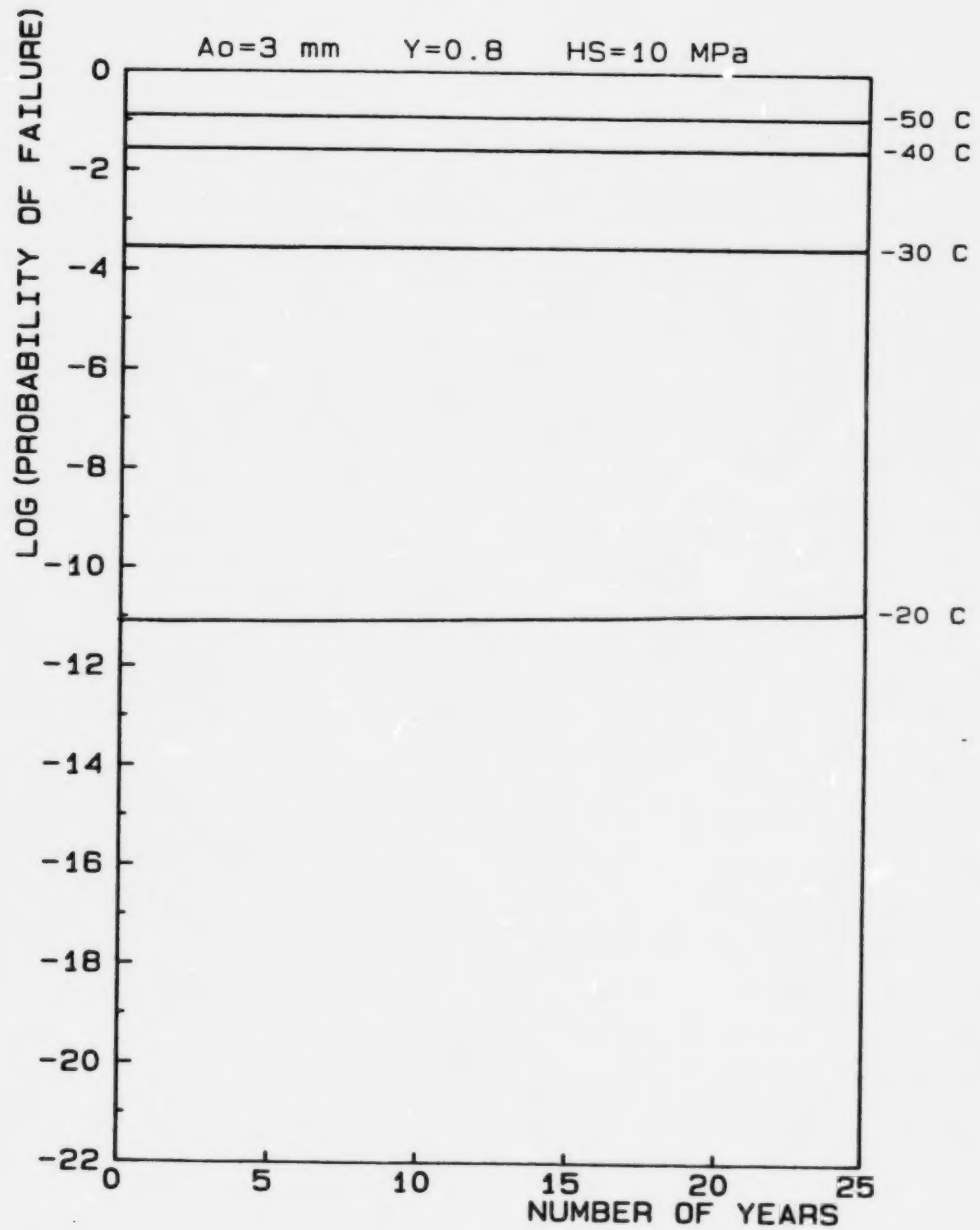


Figure C-46

- C43 -

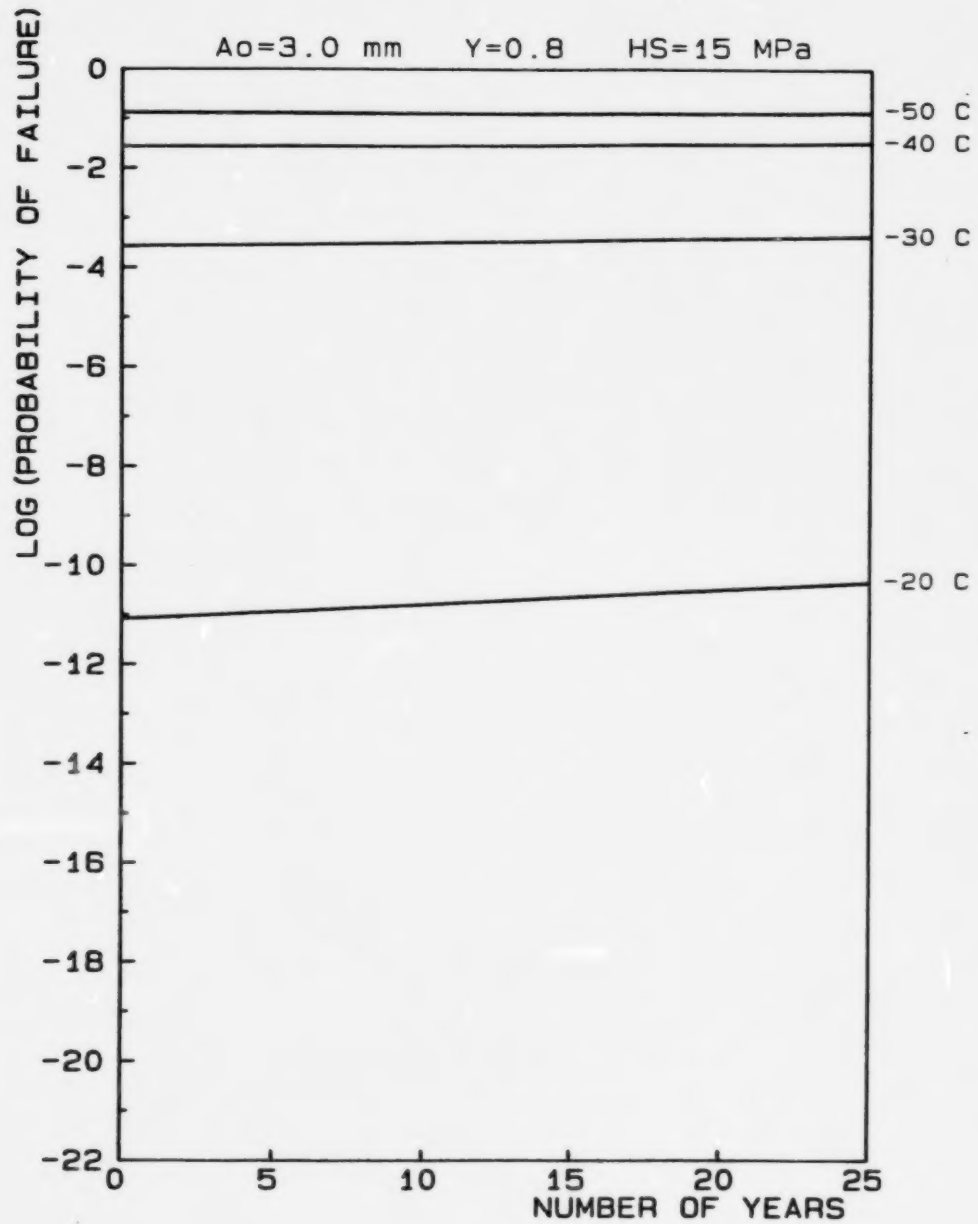


Figure C-47

- C44 -

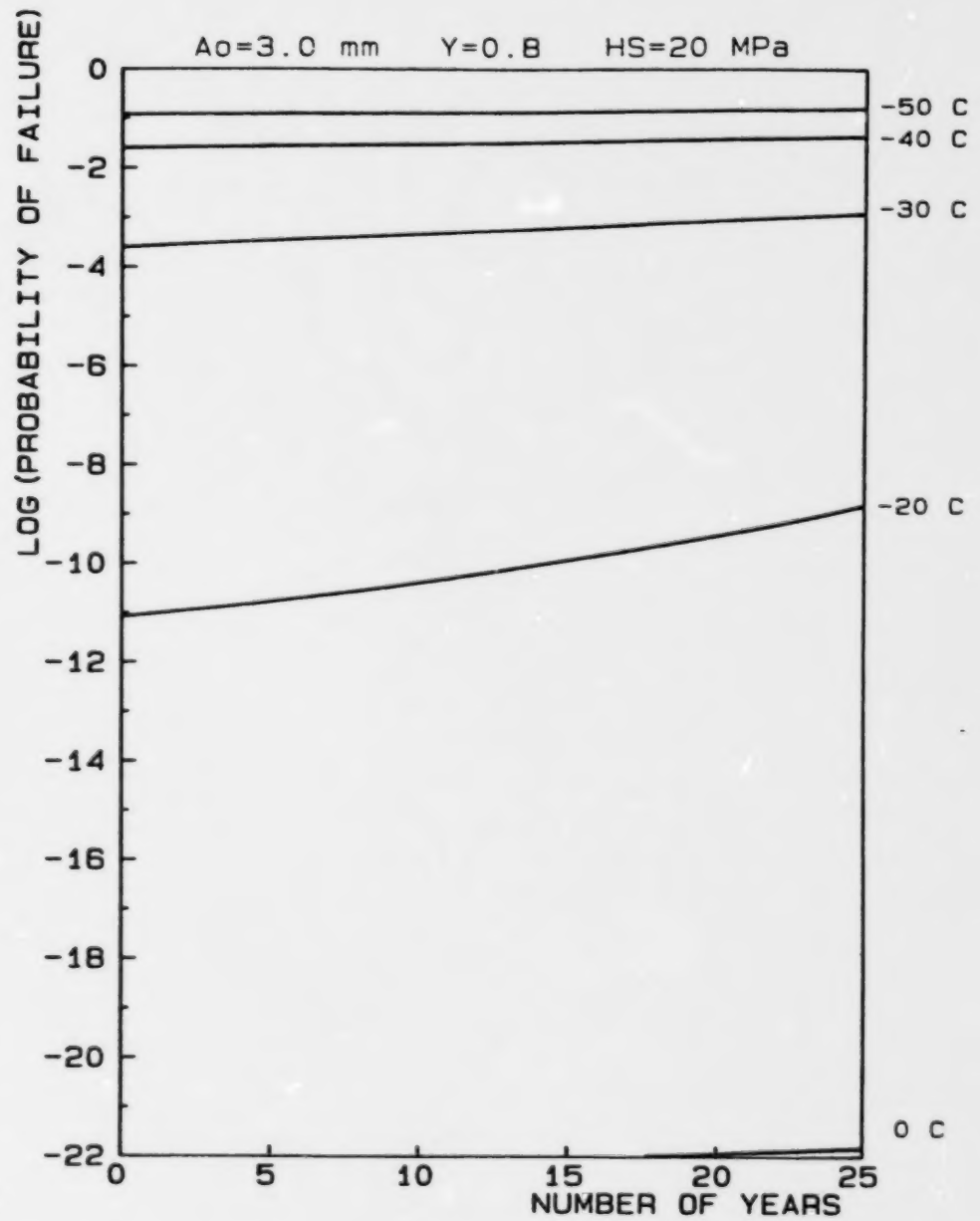


Figure C-48

- C45 -

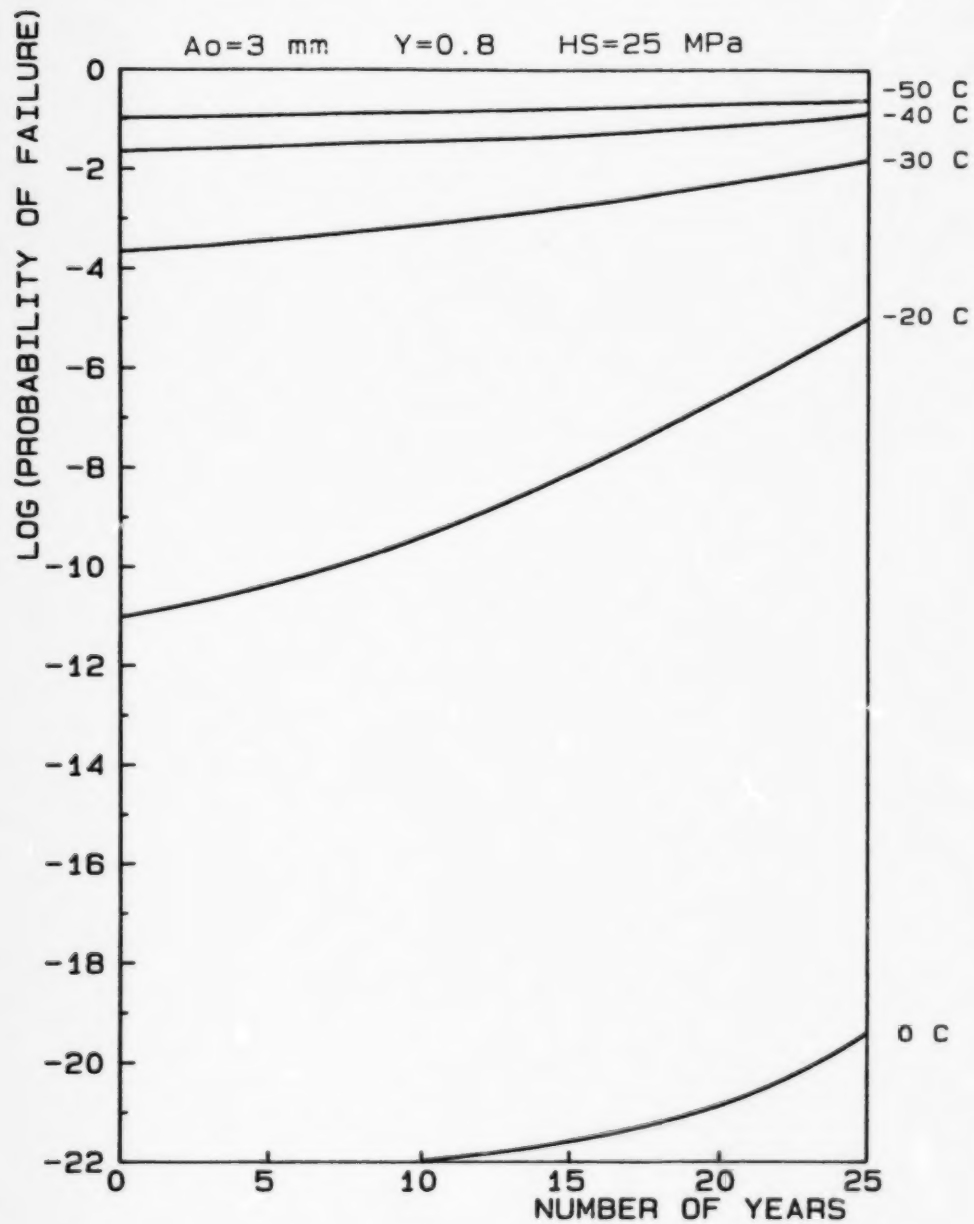


Figure C-49

- C46 -

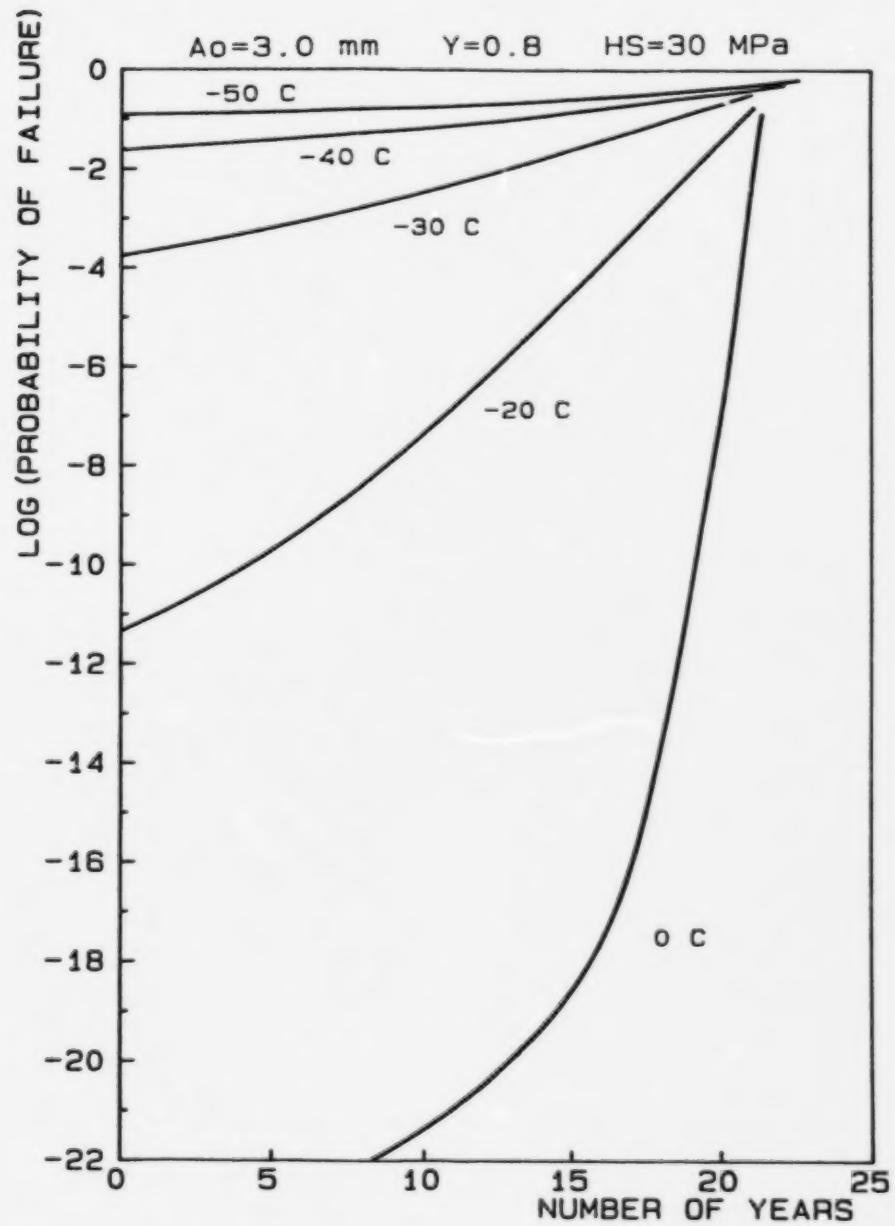


Figure C-50

- C47 -

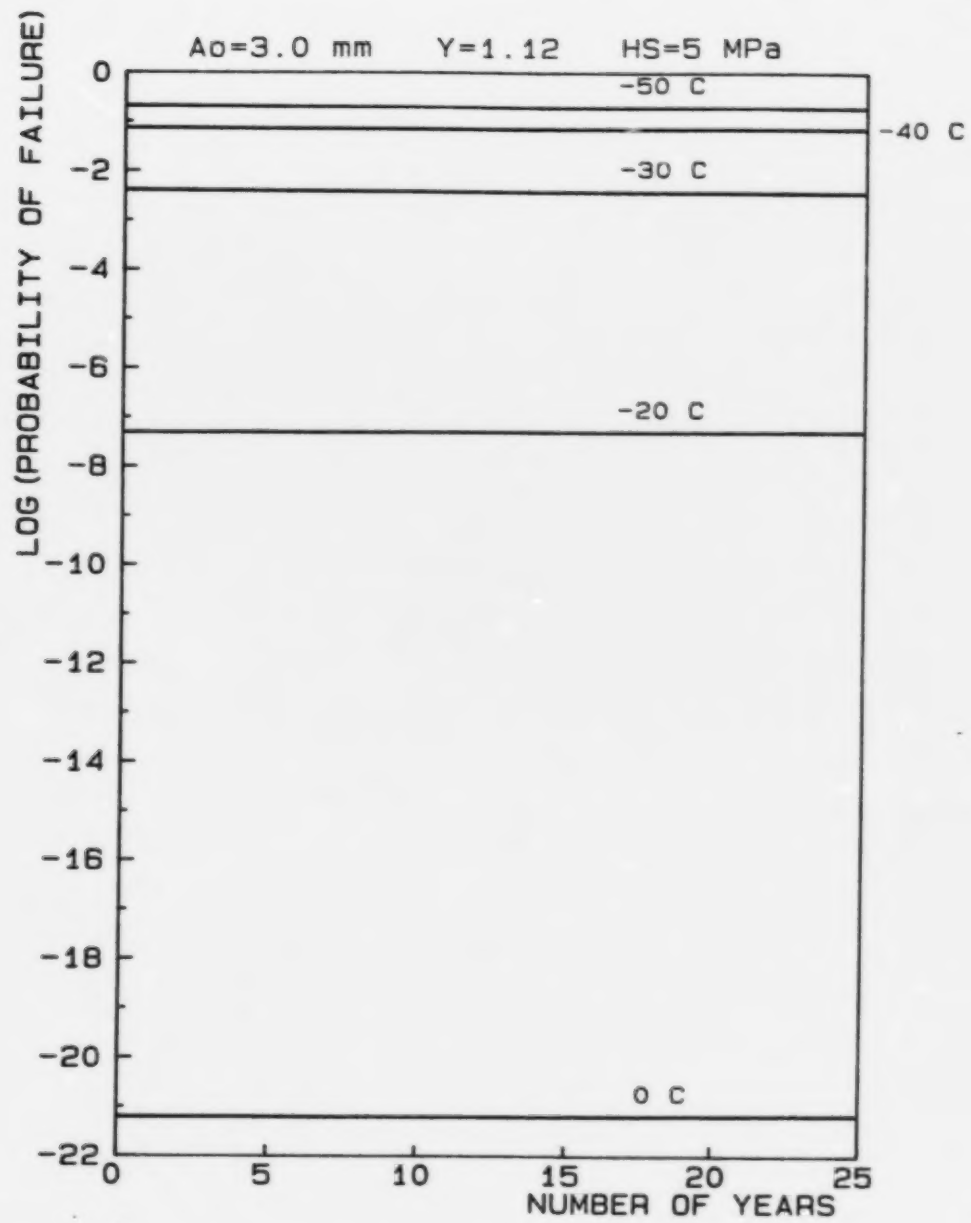


Figure C-51

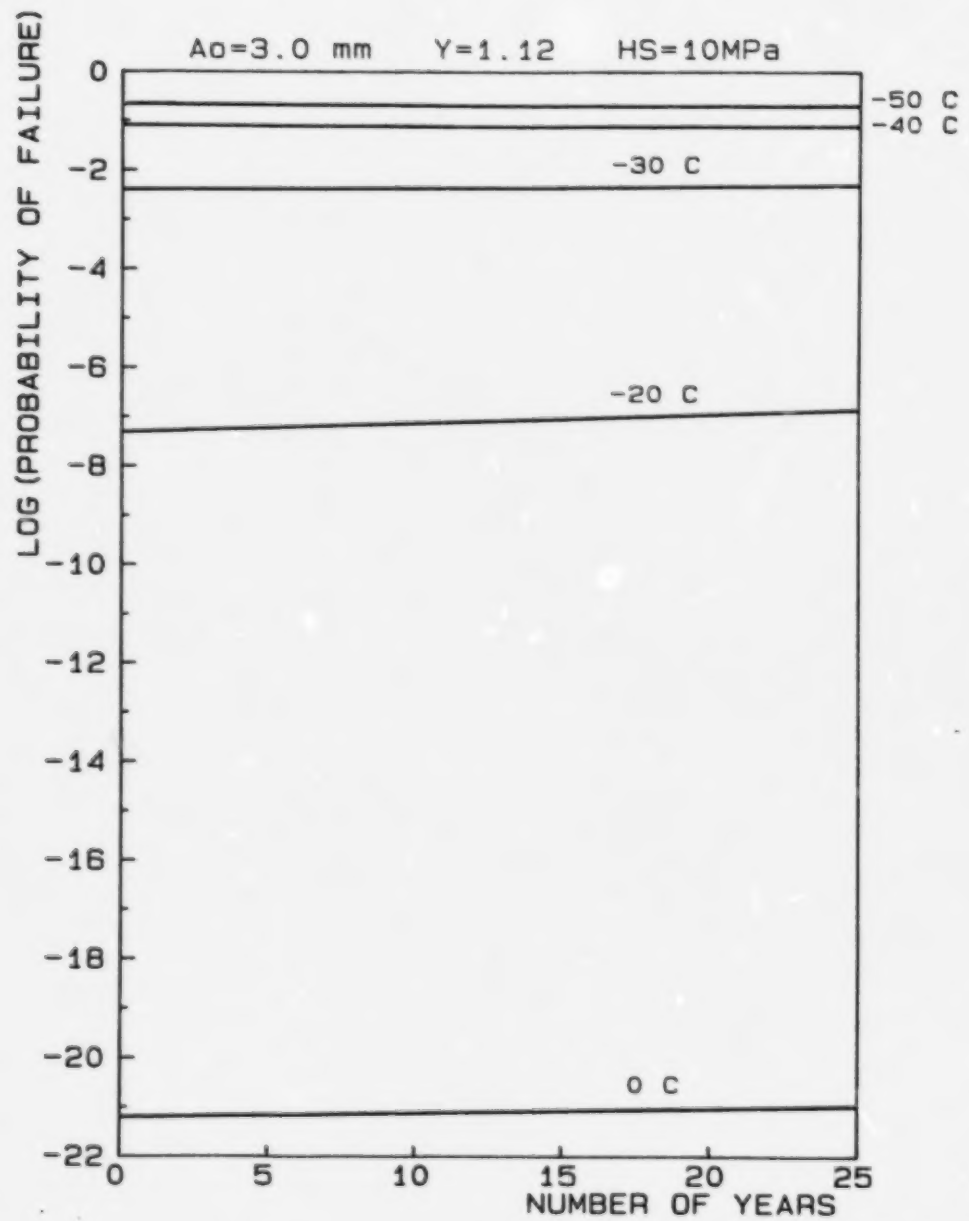


Figure C-52

- C49 -

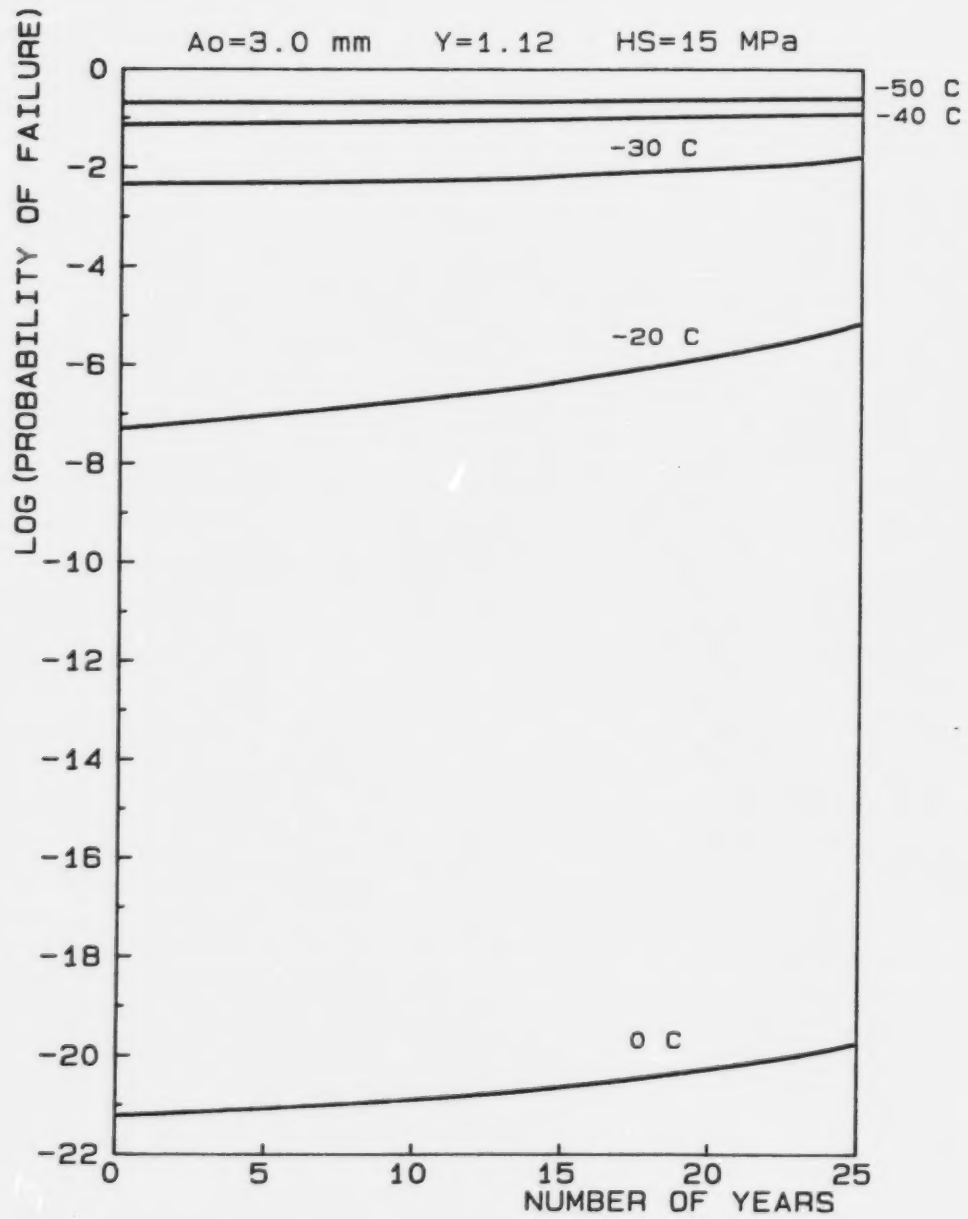


Figure C-53

- C50 -

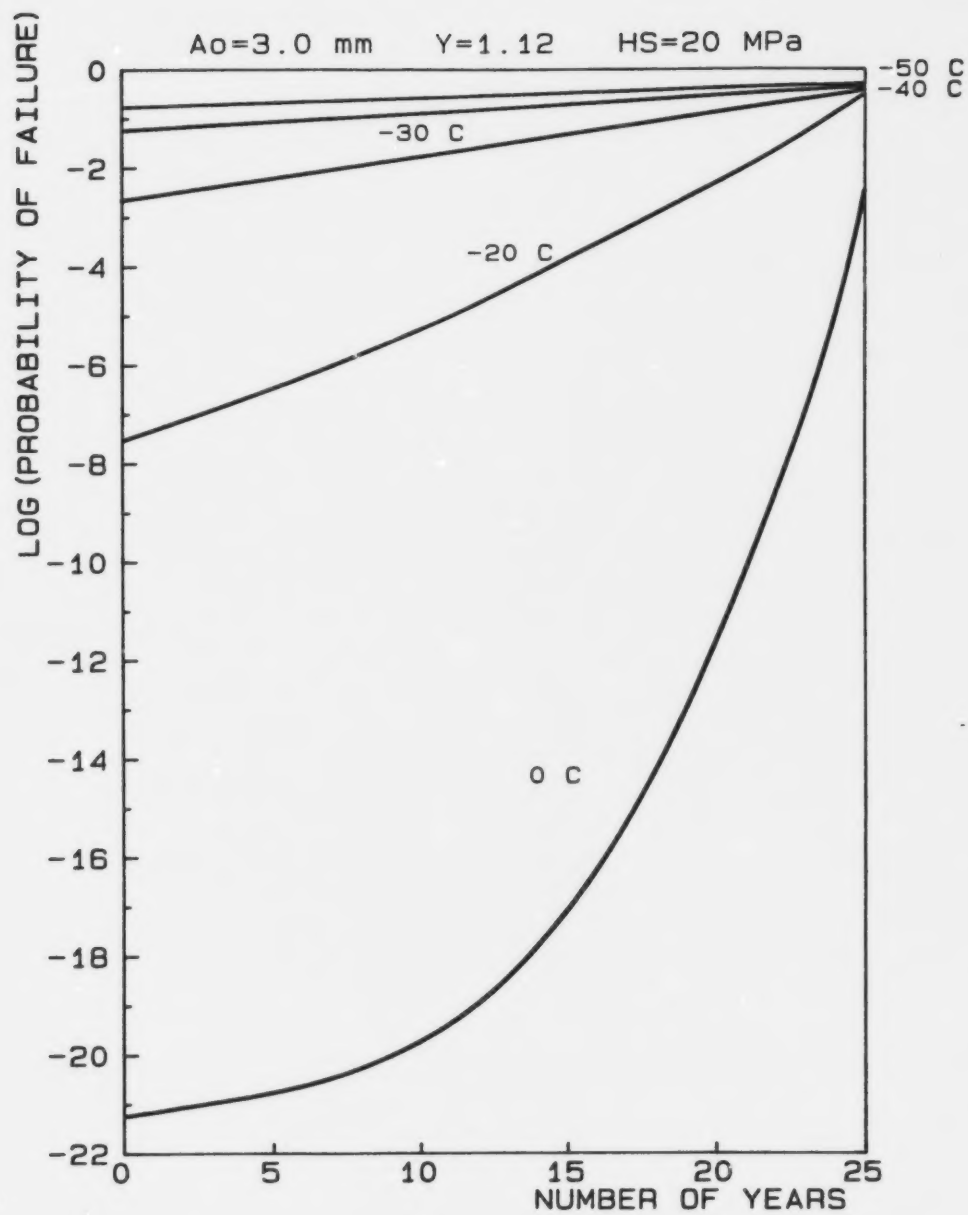


Figure C-54

- C51 -

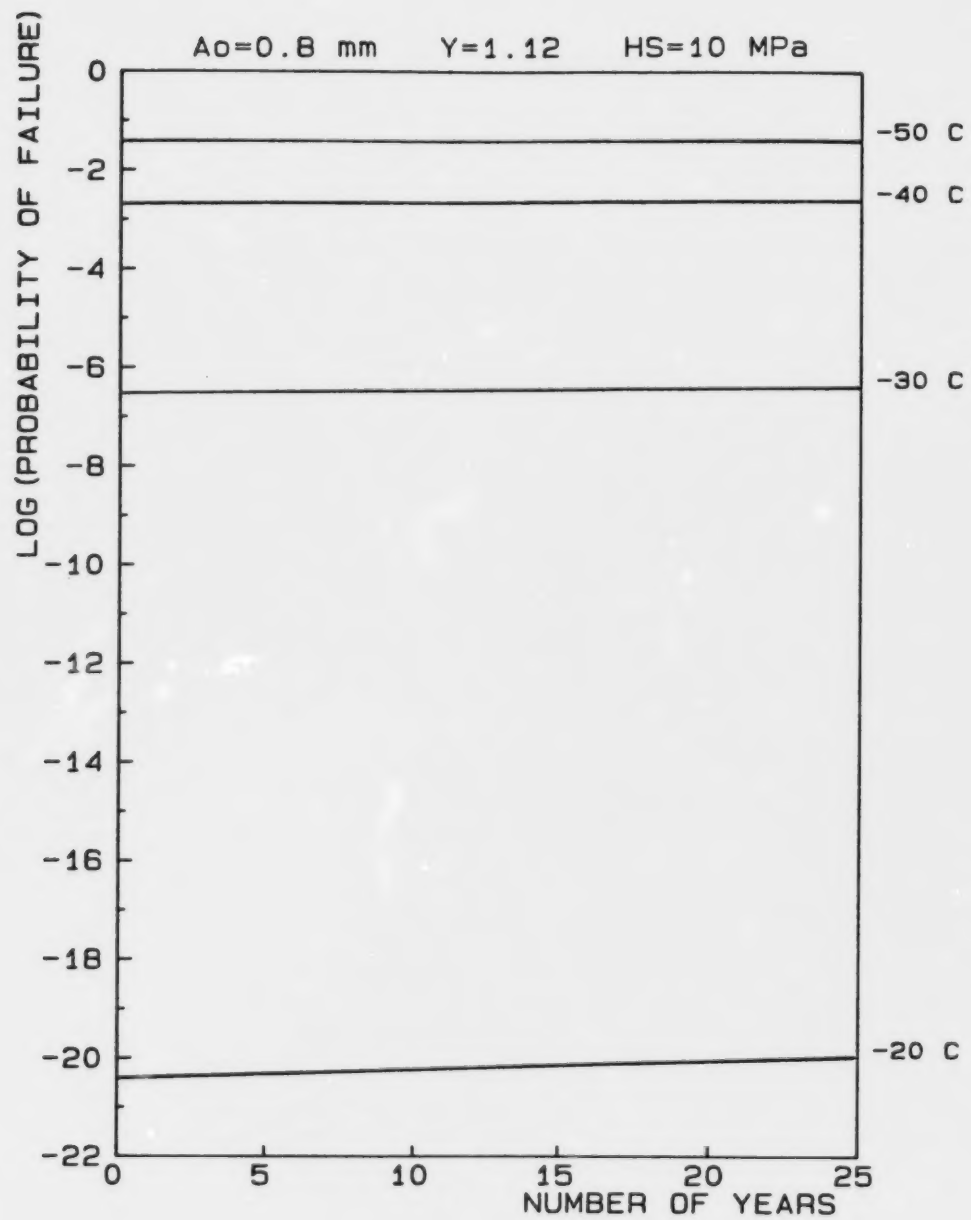


Figure C-55 to C-66

Probability of Failure as a Function of Time
T-Joint, $\Phi = 0.50$

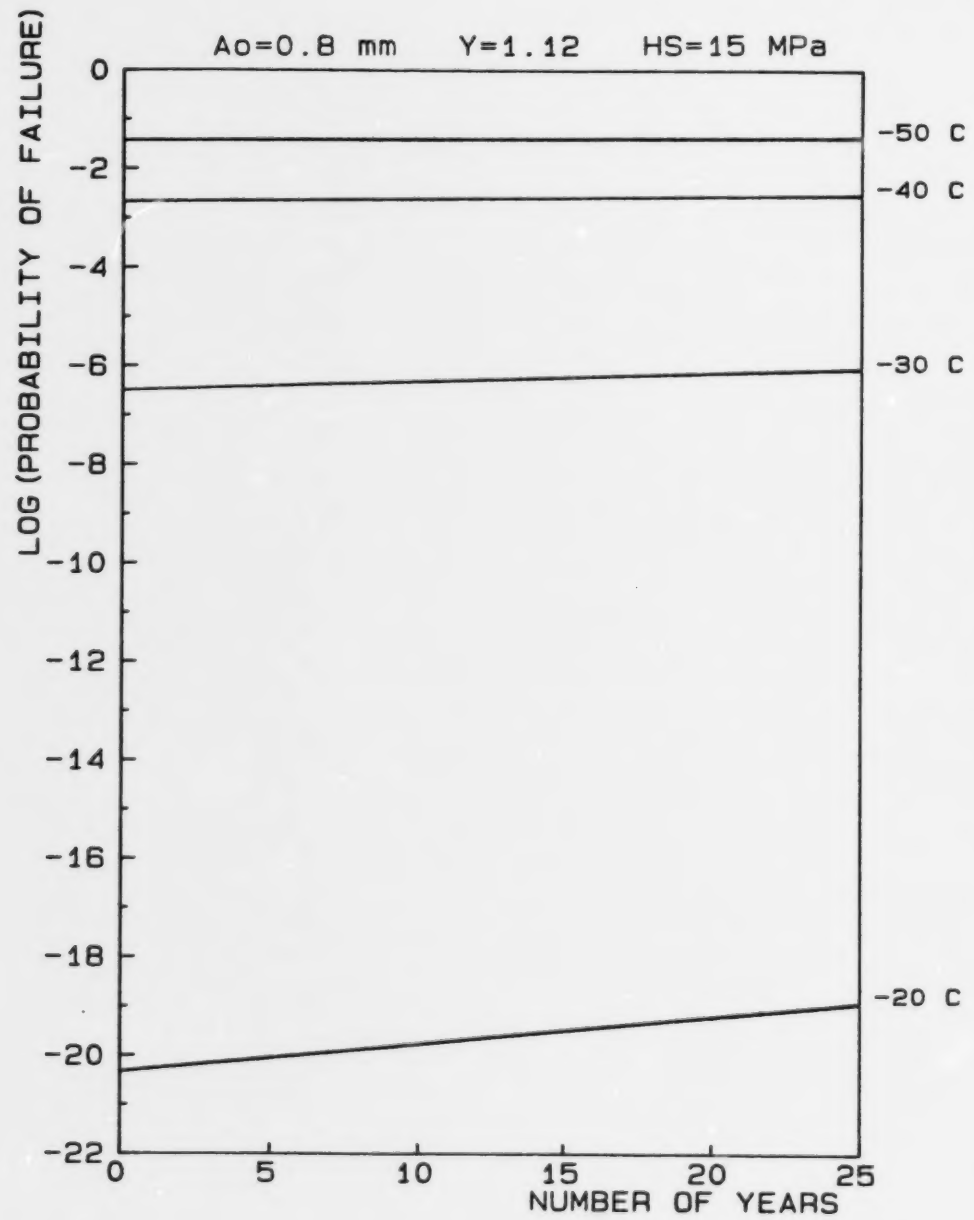


Figure C-56

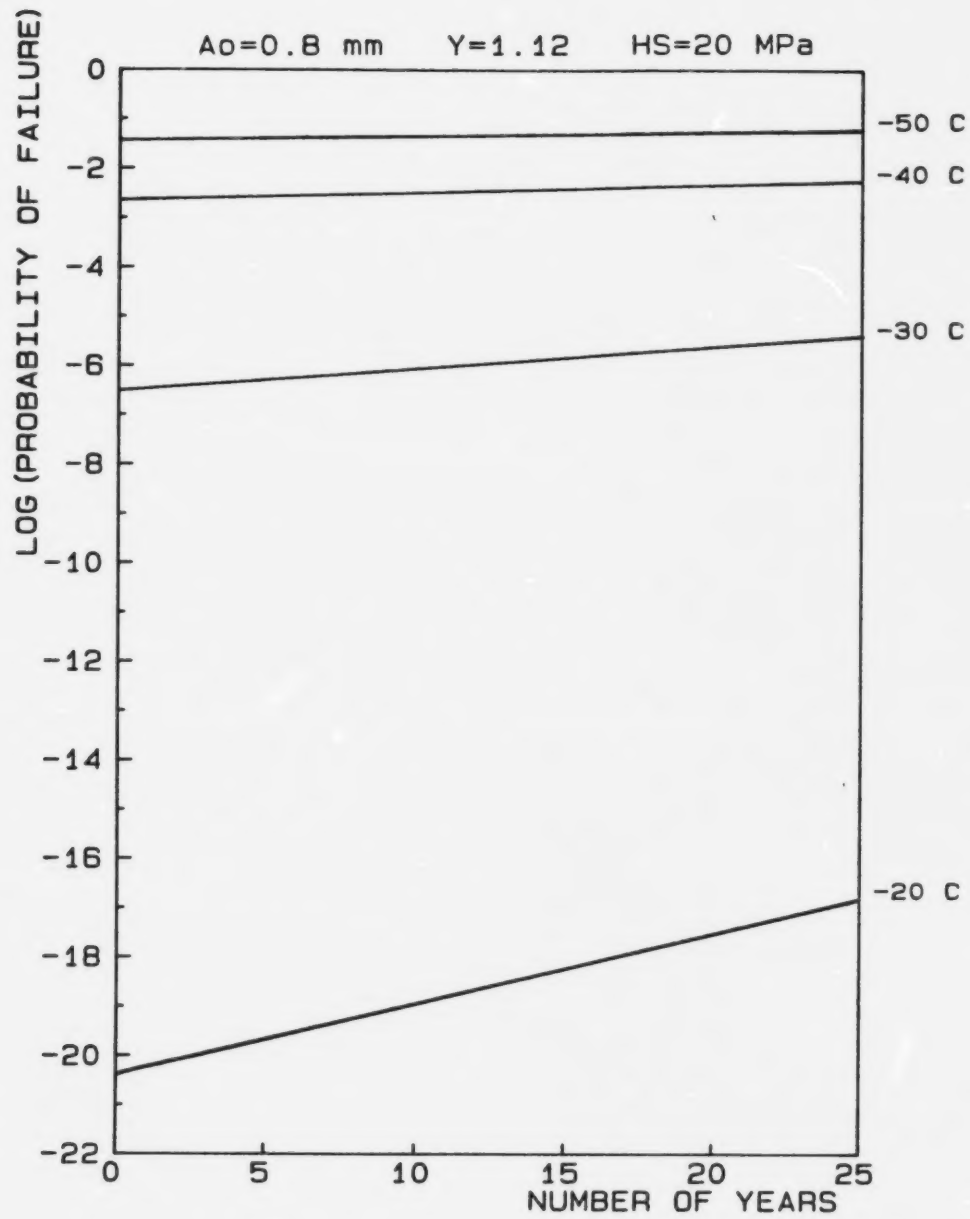


Figure C-57

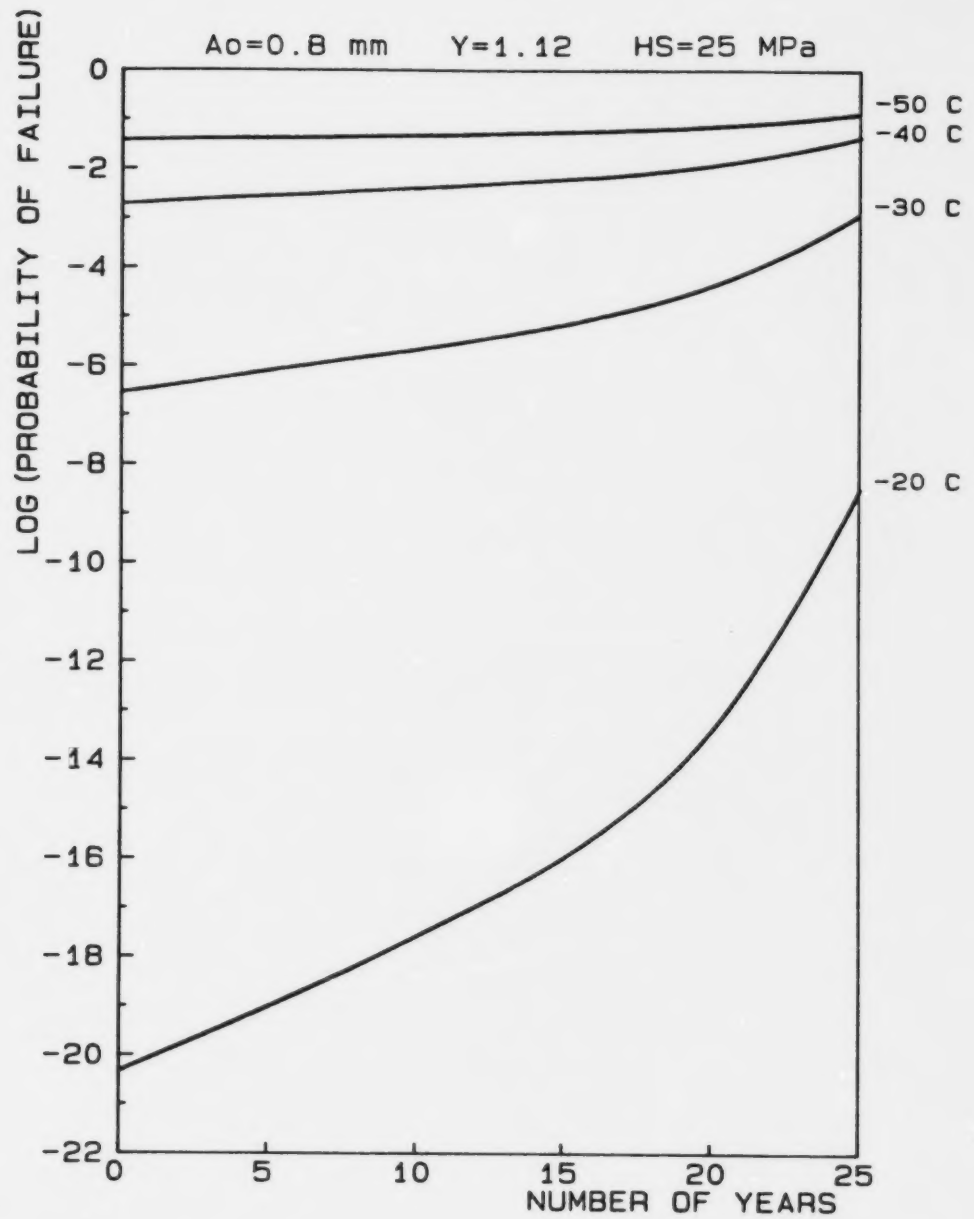


Figure C-58

- C55 -

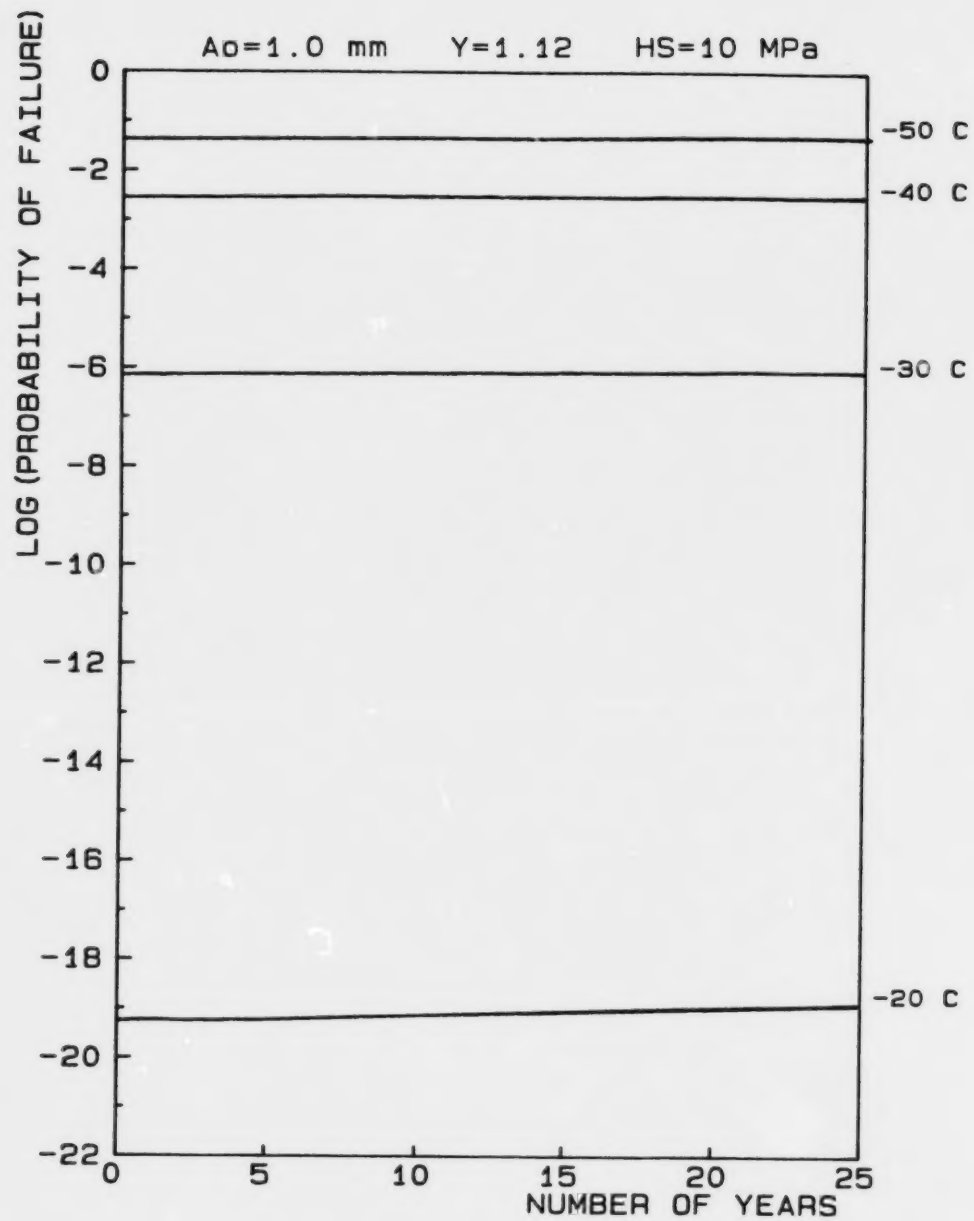


Figure C-59

- C56 -

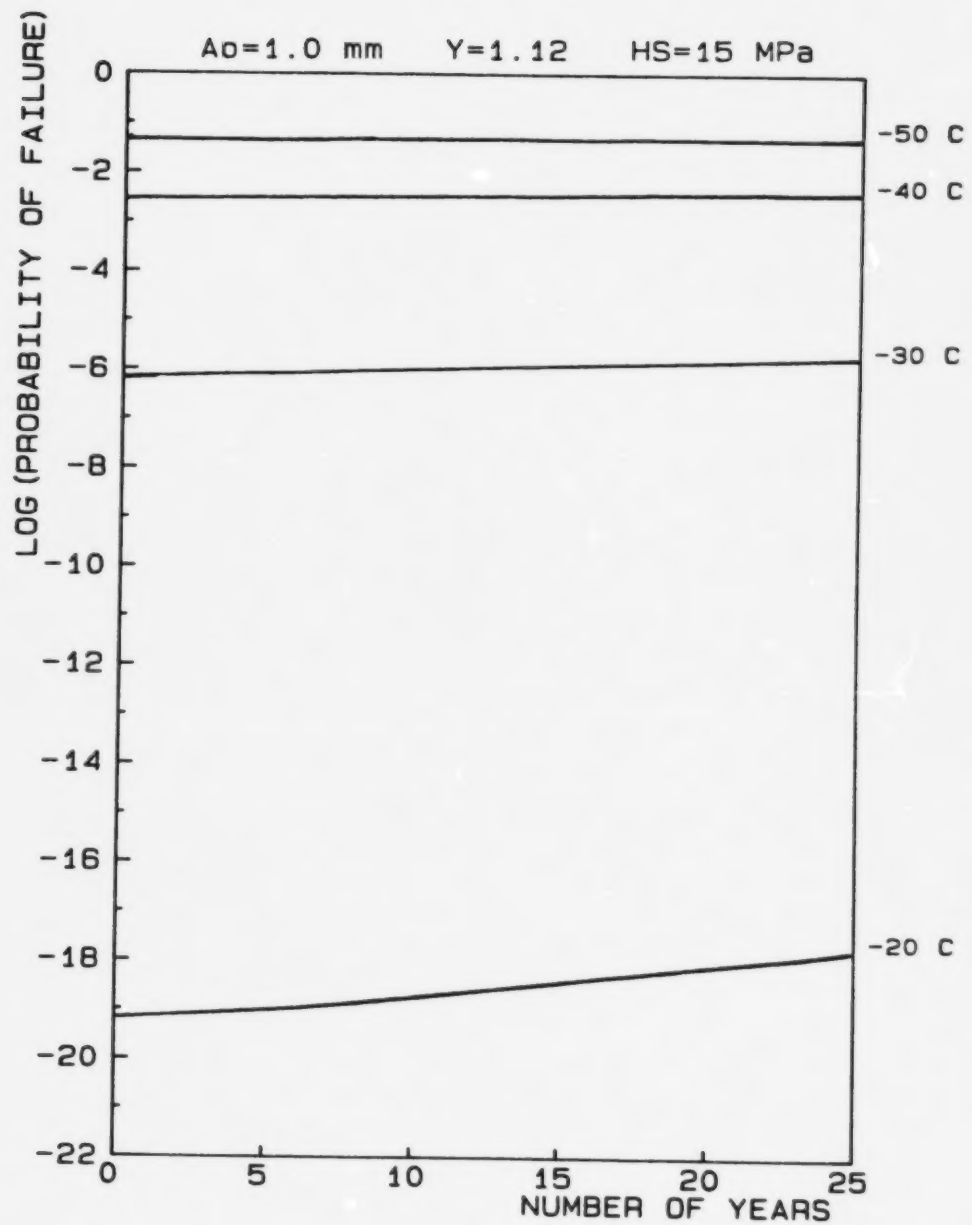


Figure C-60

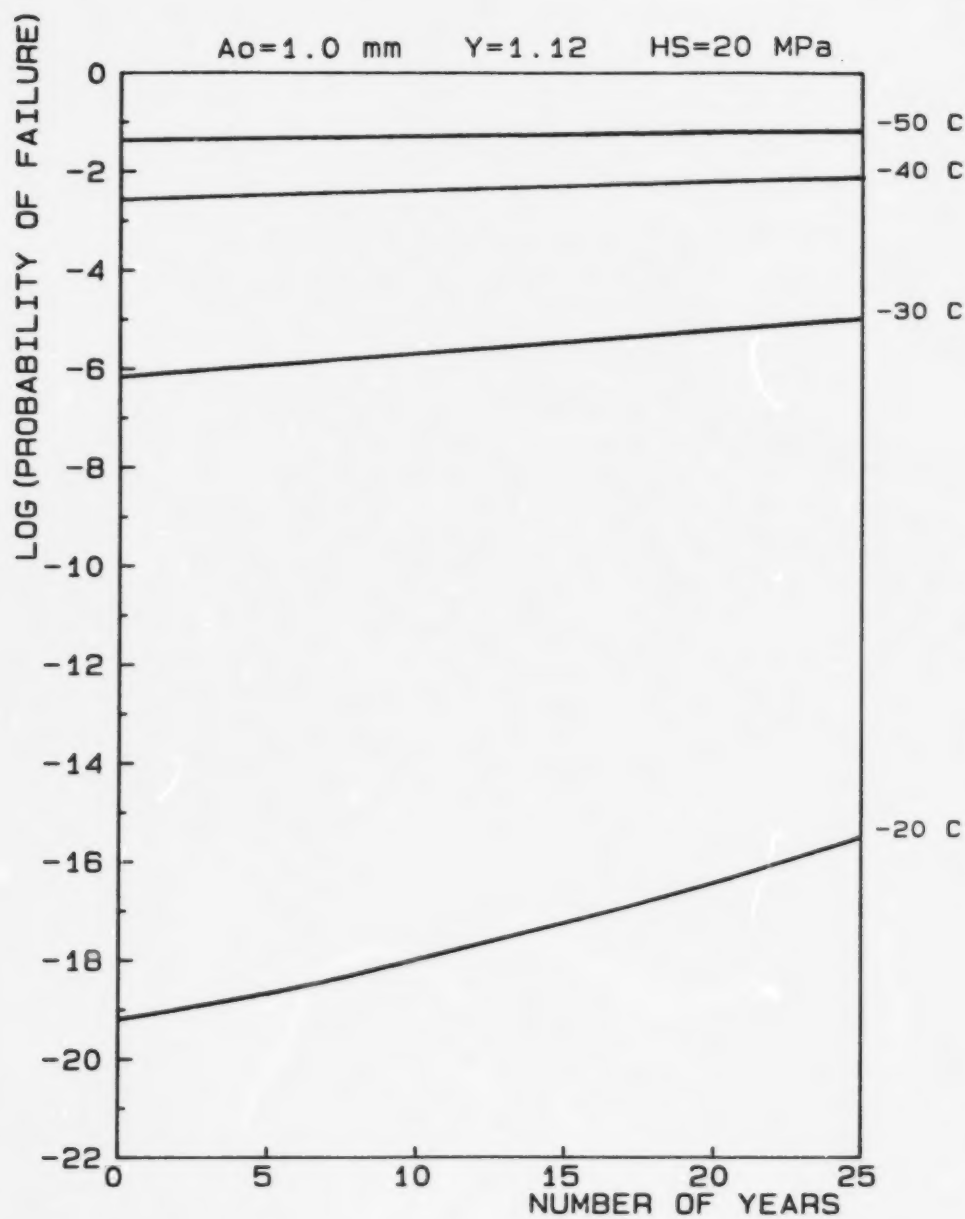


Figure C-61

- C58 -

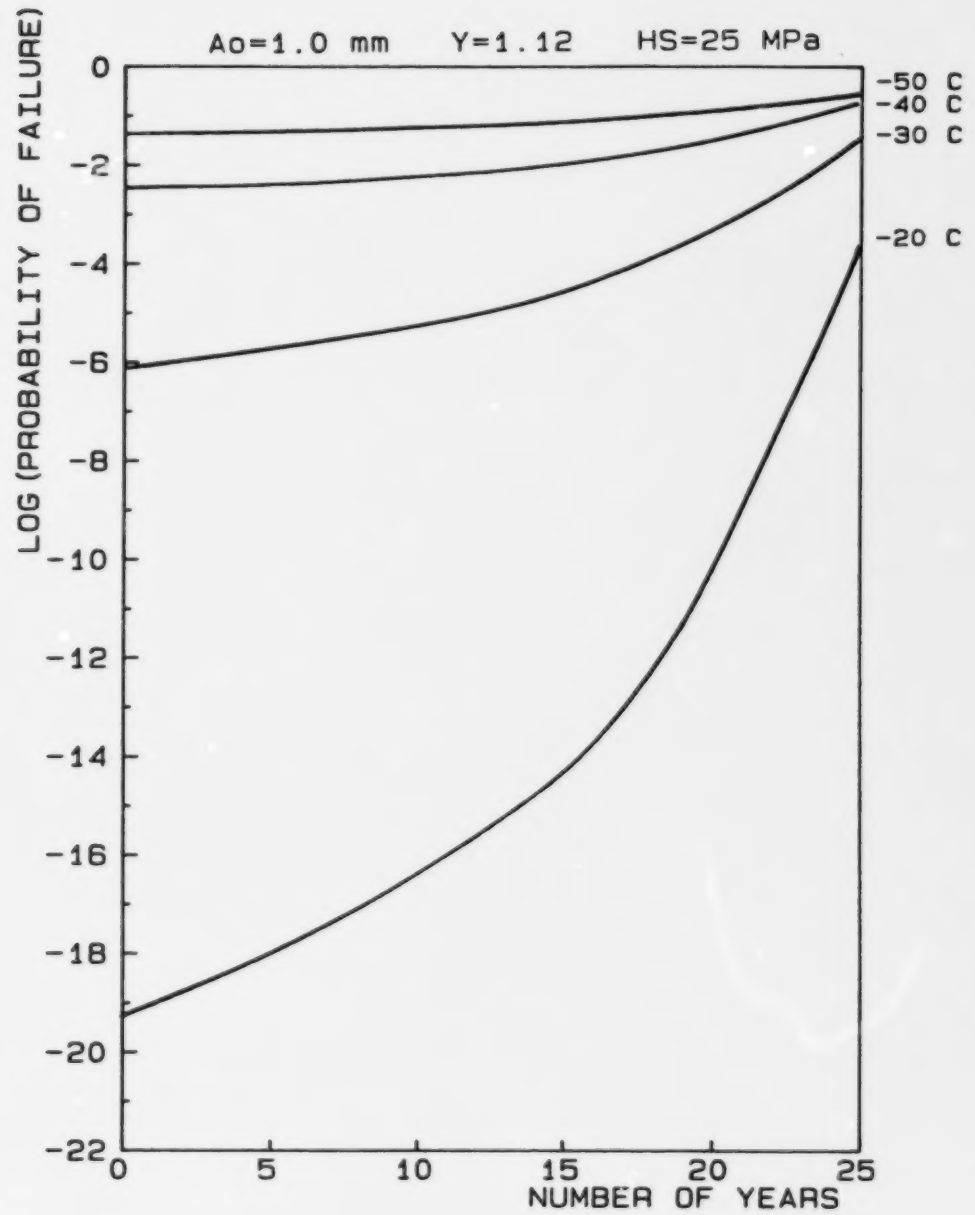


Figure C-62

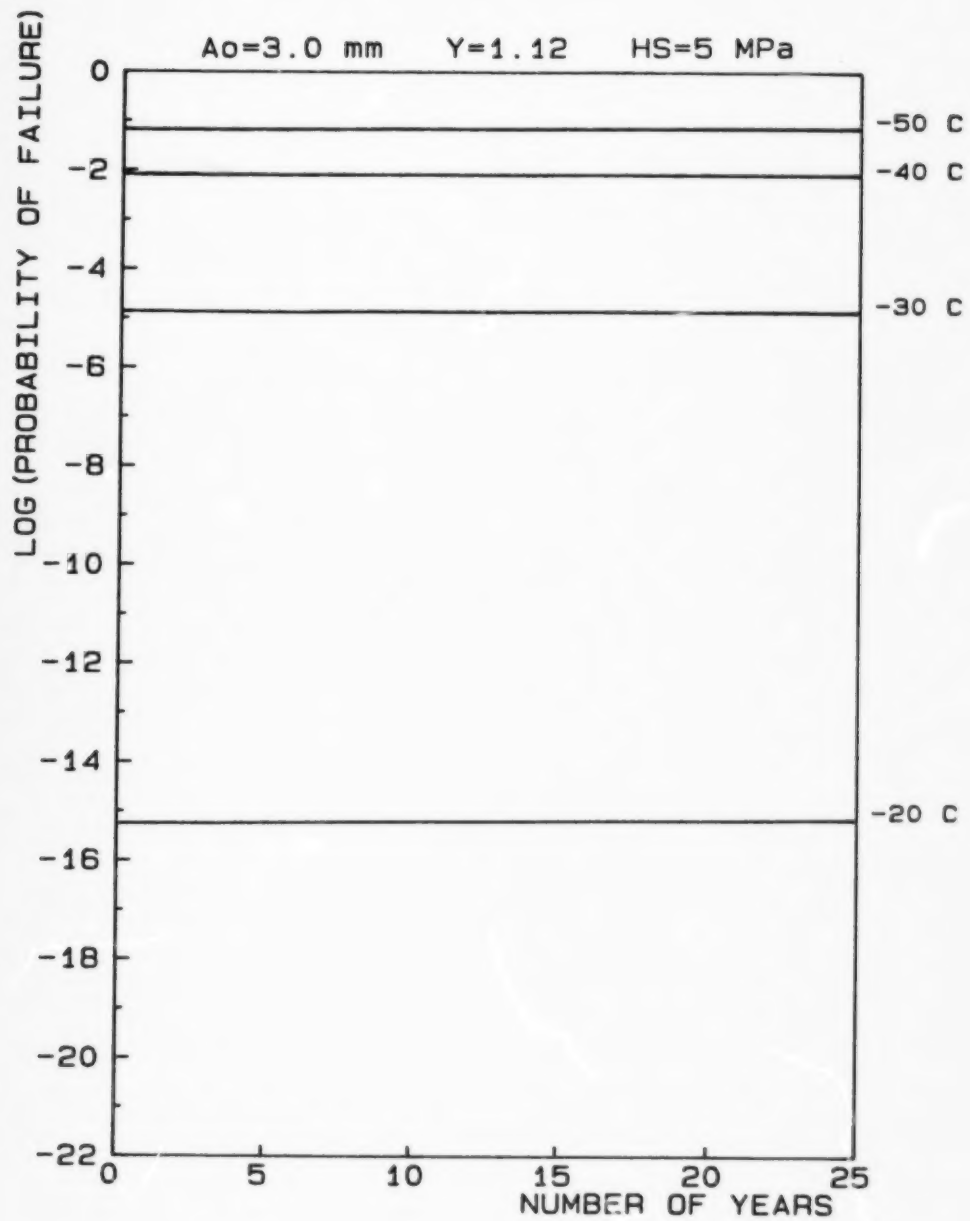


Figure C-63

- C60 -

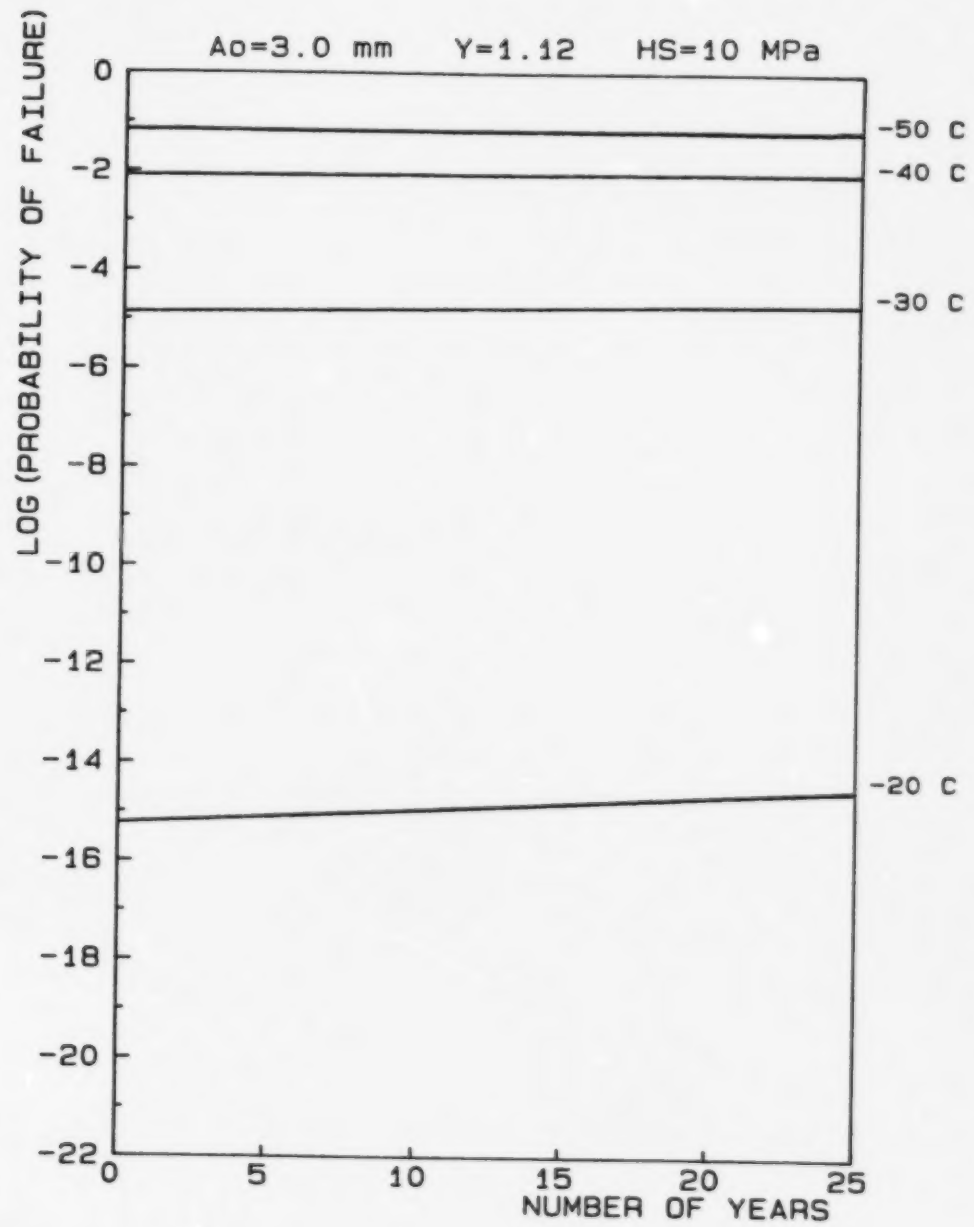


Figure C-64

- C61 -

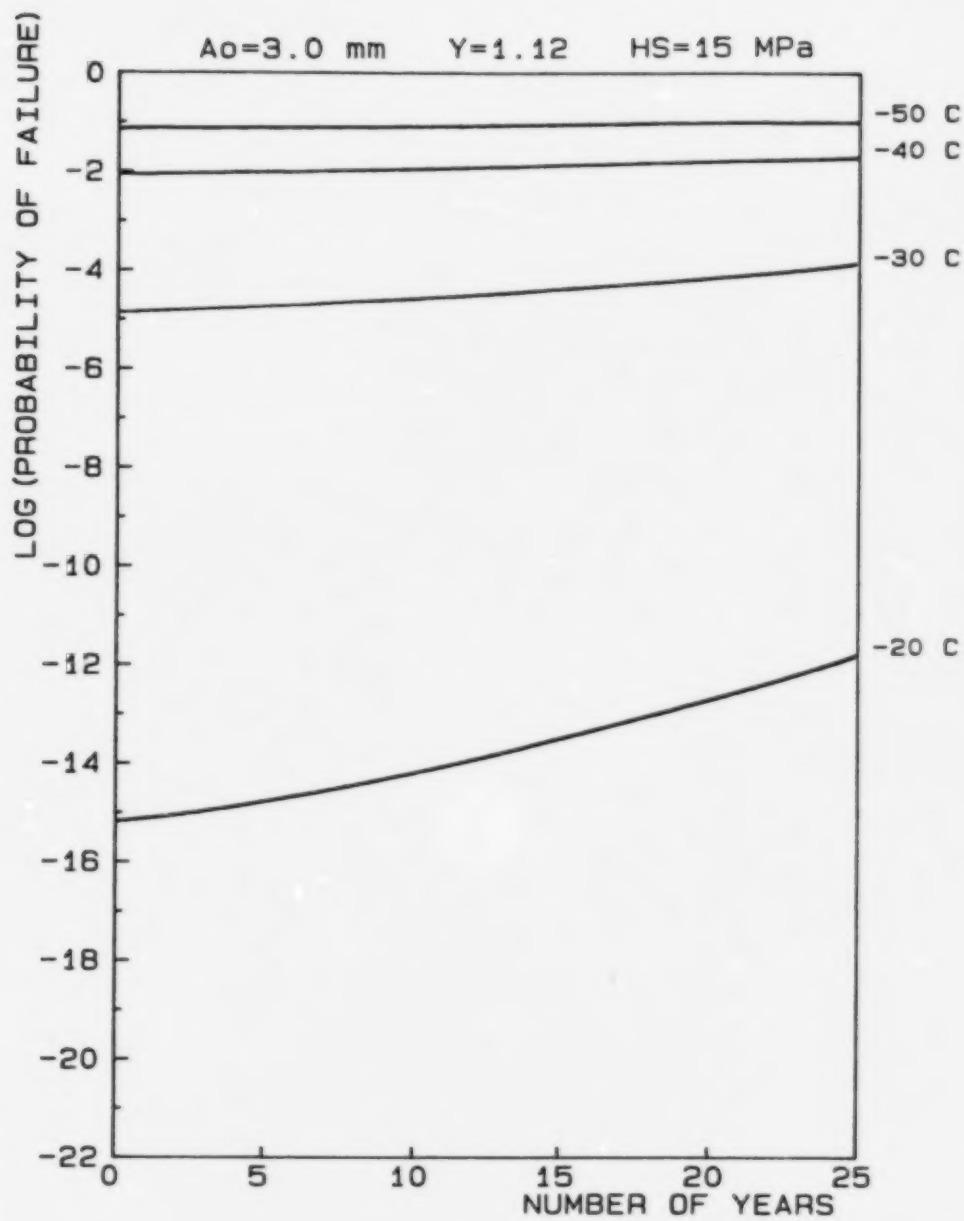


Figure C-65

- C62 -

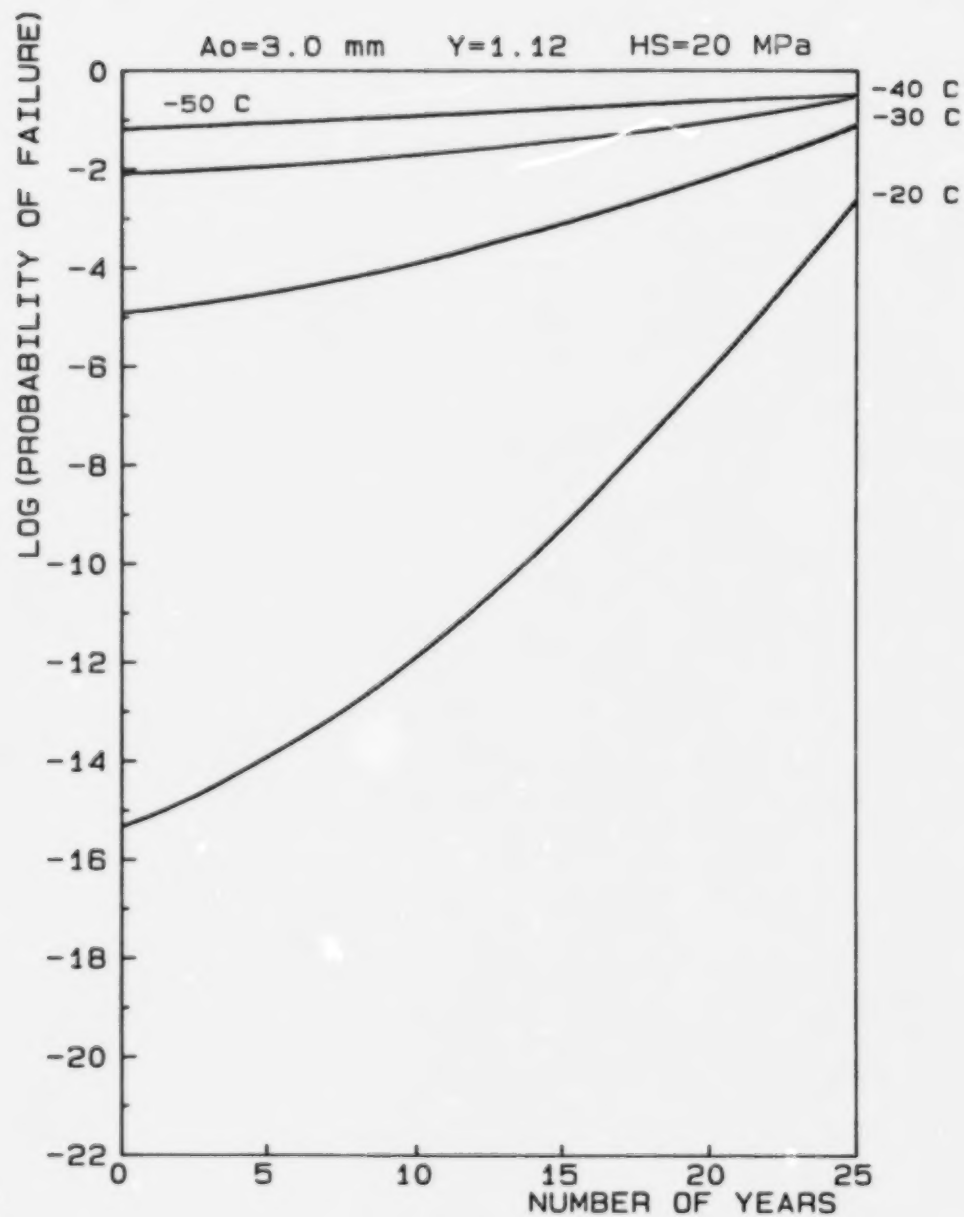


Figure C-66

- C63 -

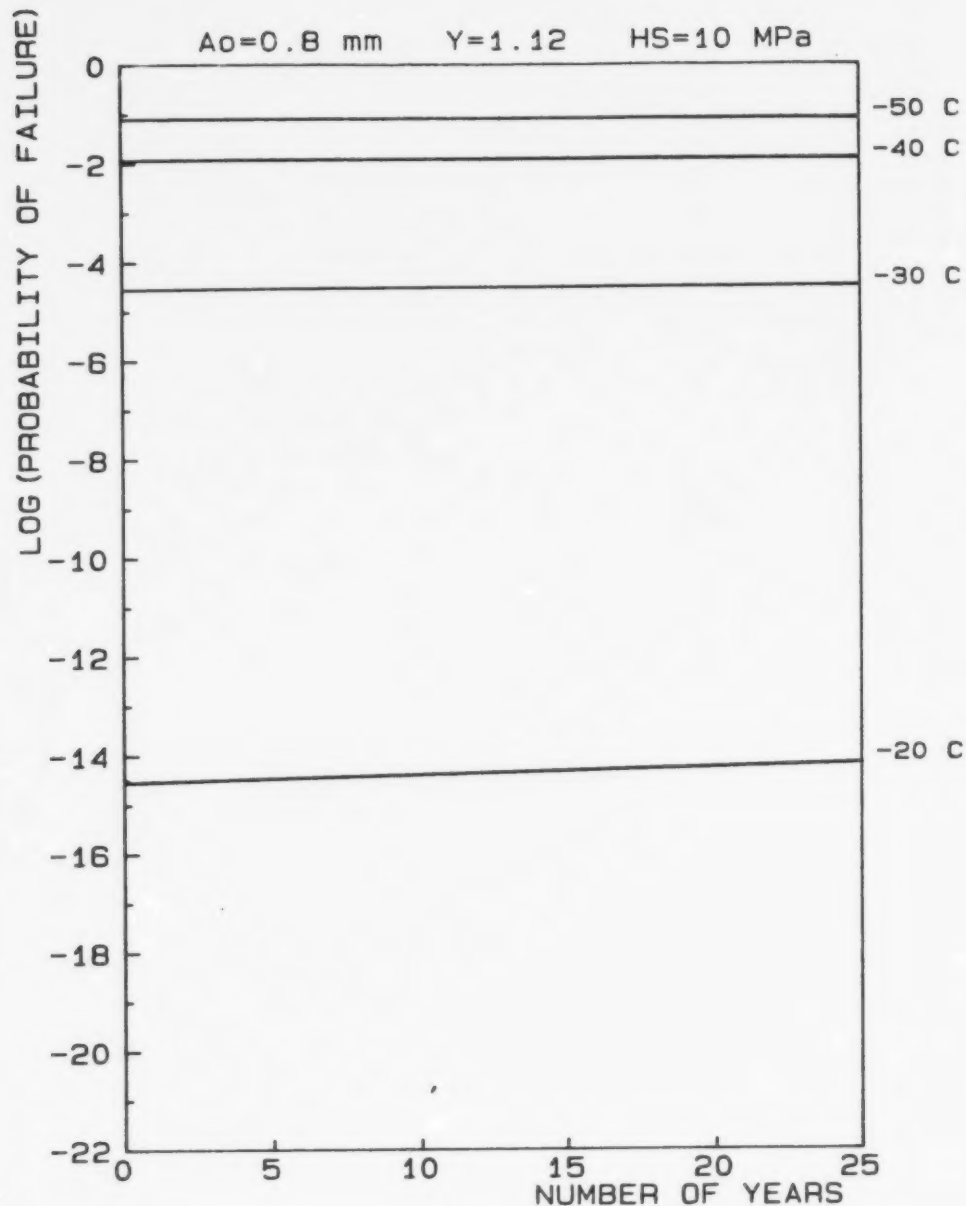


Figure C-67 to C-78

Probability of Failure as a Function of Time
Y-Joint, $\Phi = 0.05$

- C64 -

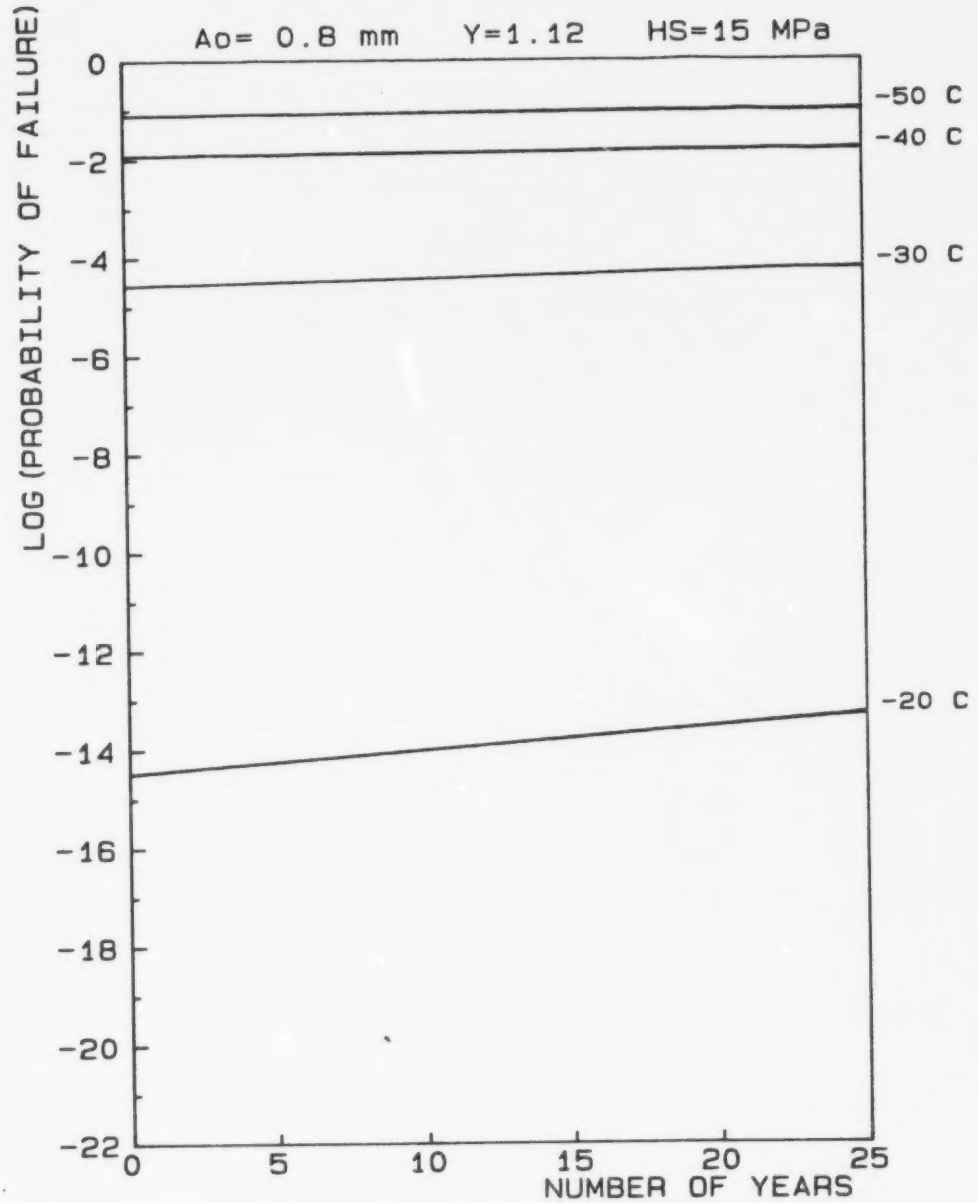


Figure C-68

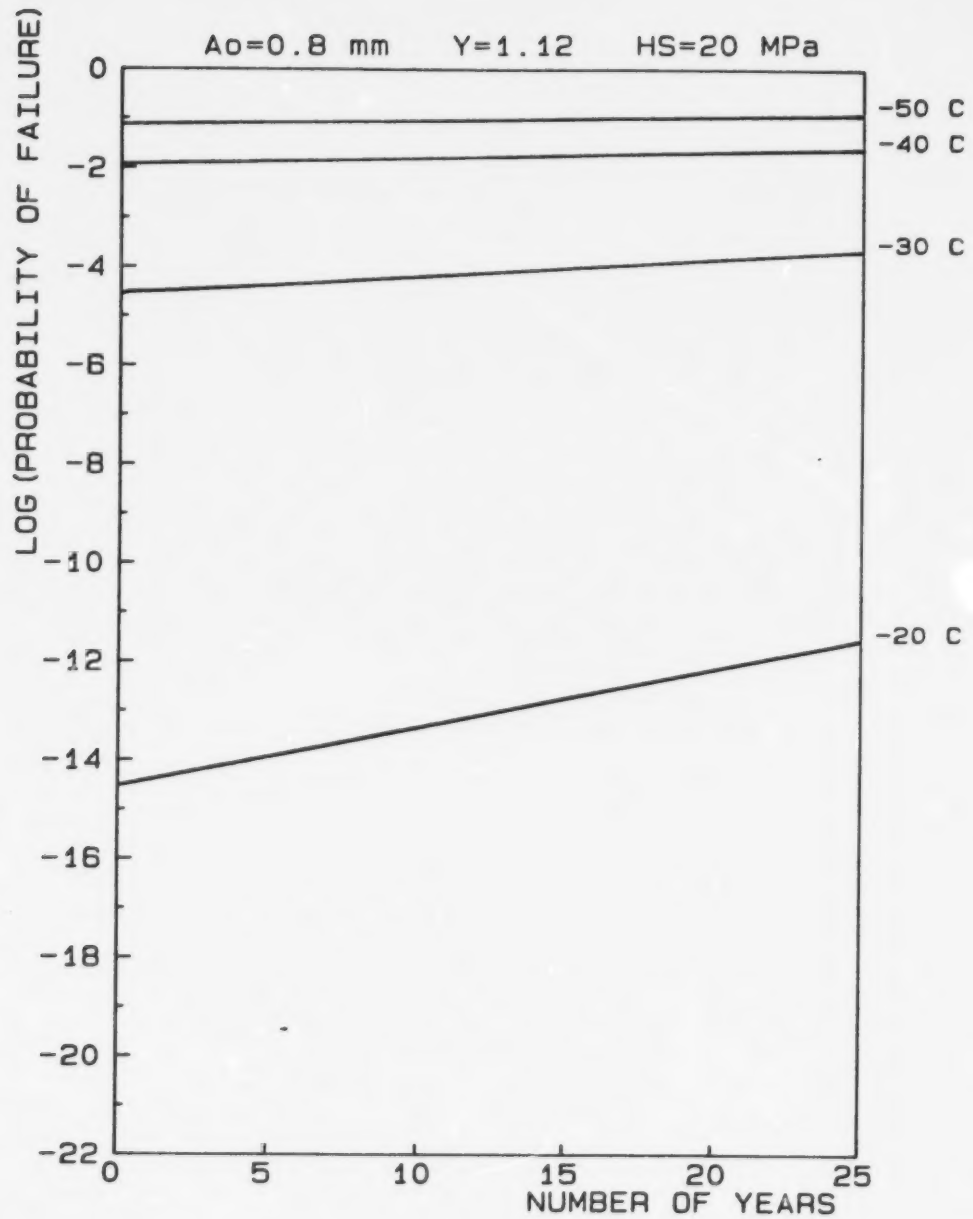


Figure C-69

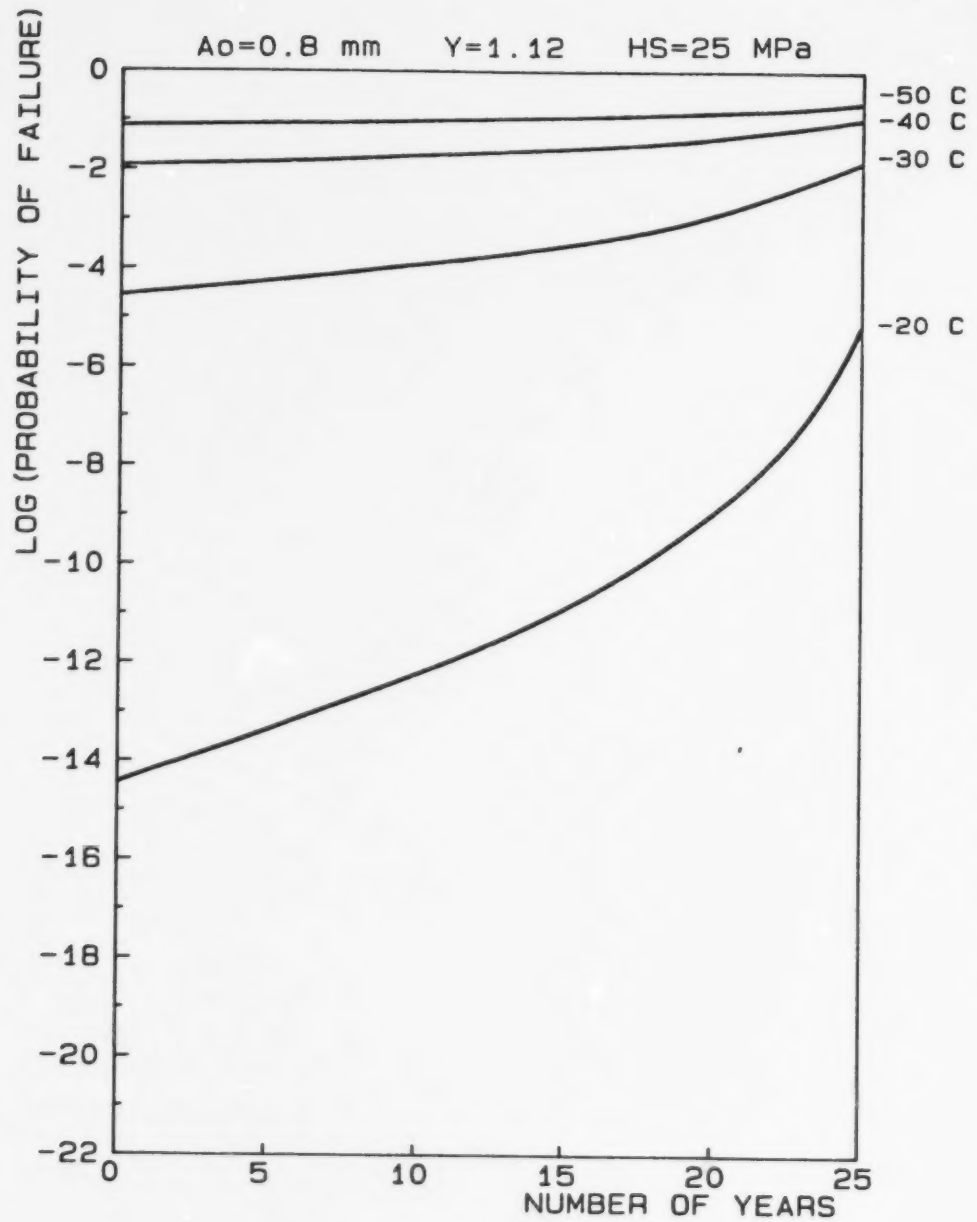


Figure C-70

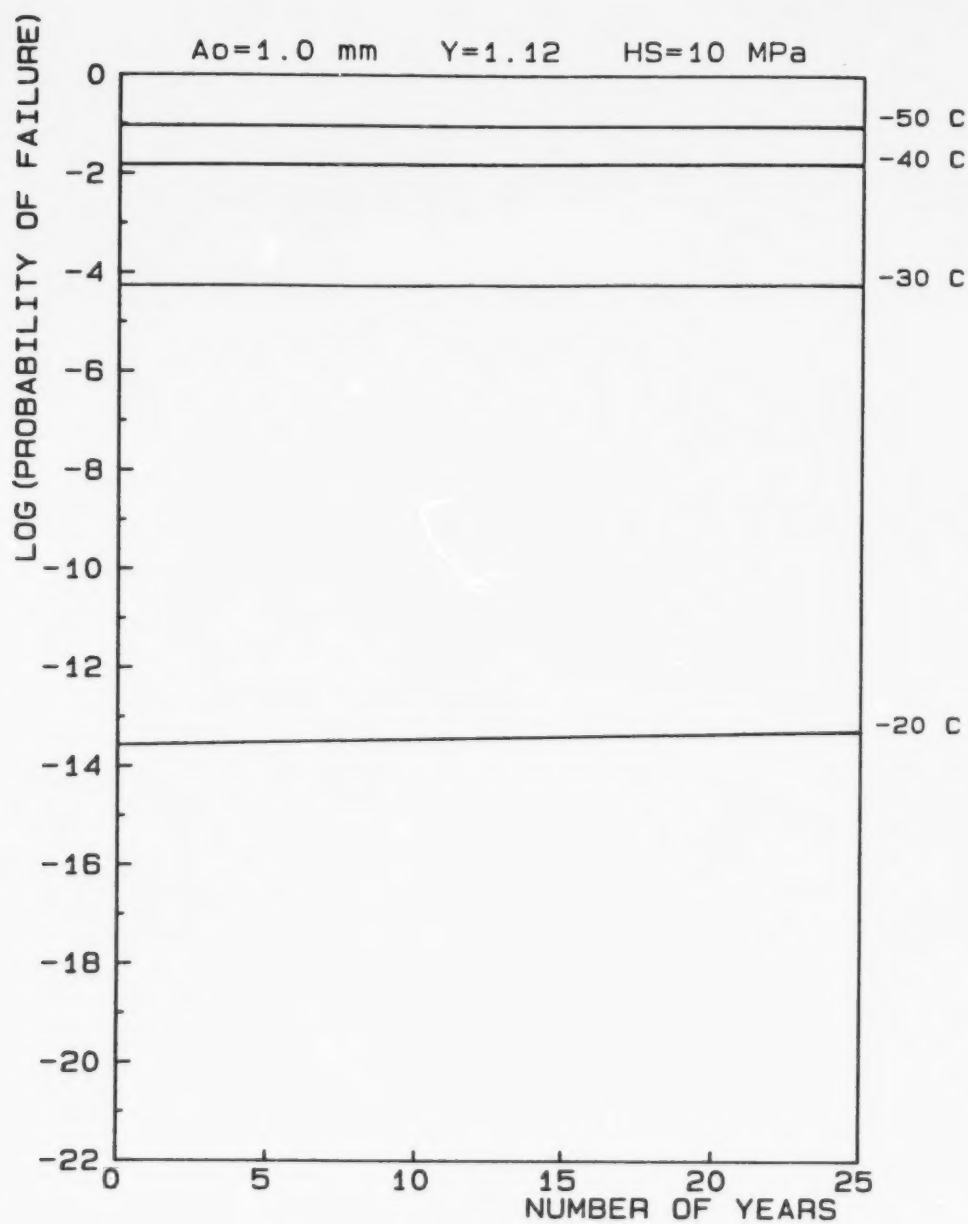


Figure C-71

- C68 -

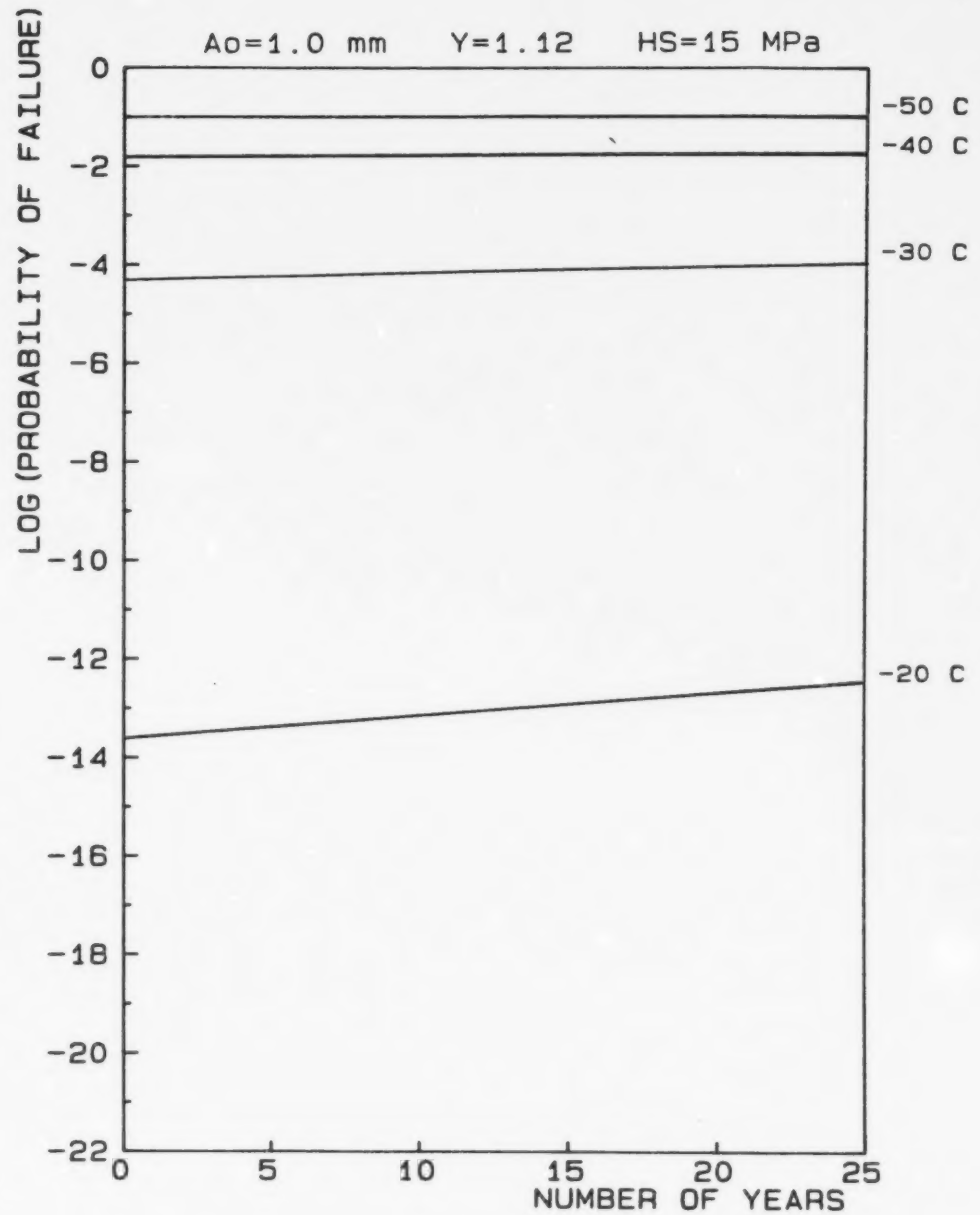


Figure C-72

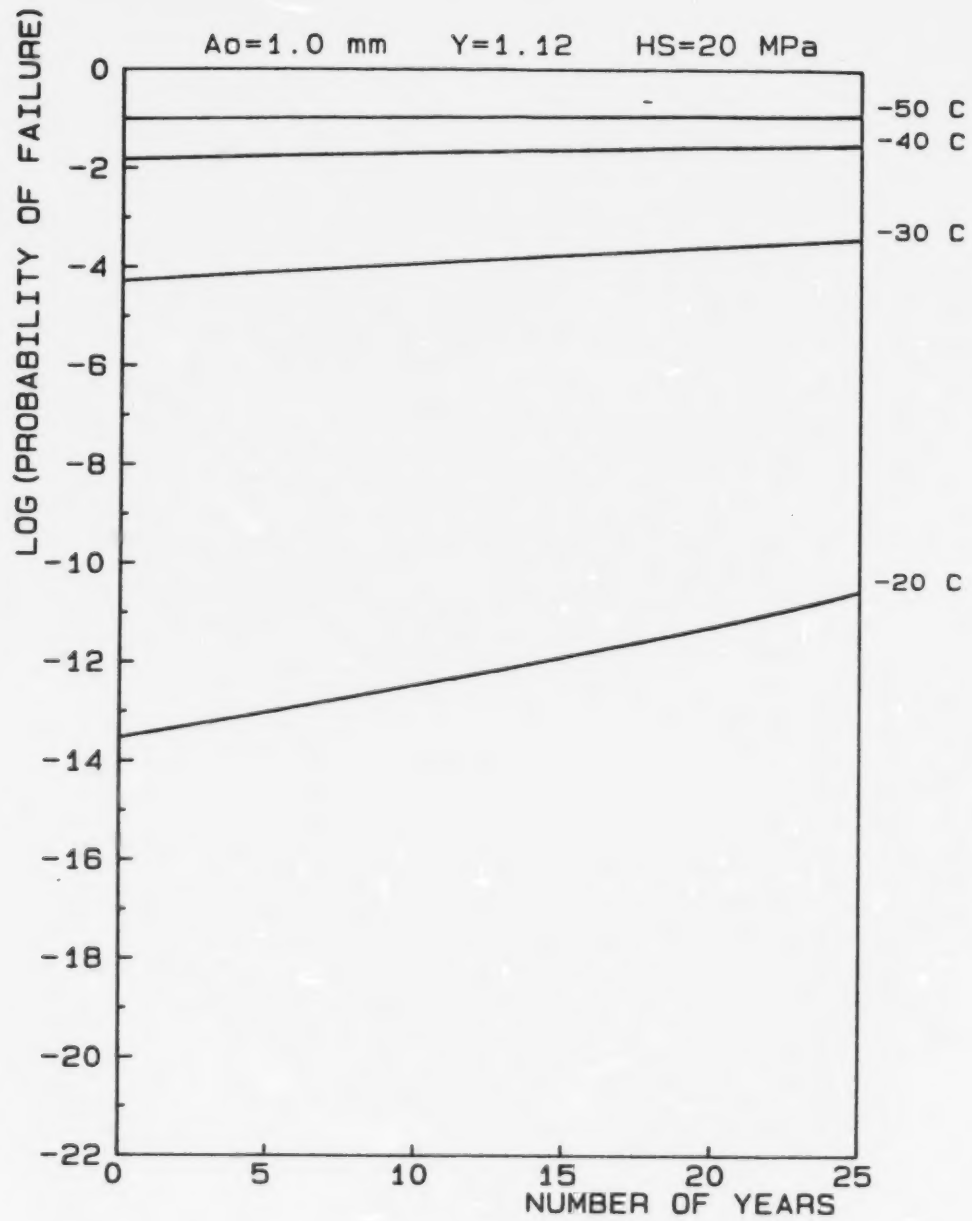


Figure C-73

- C70 -

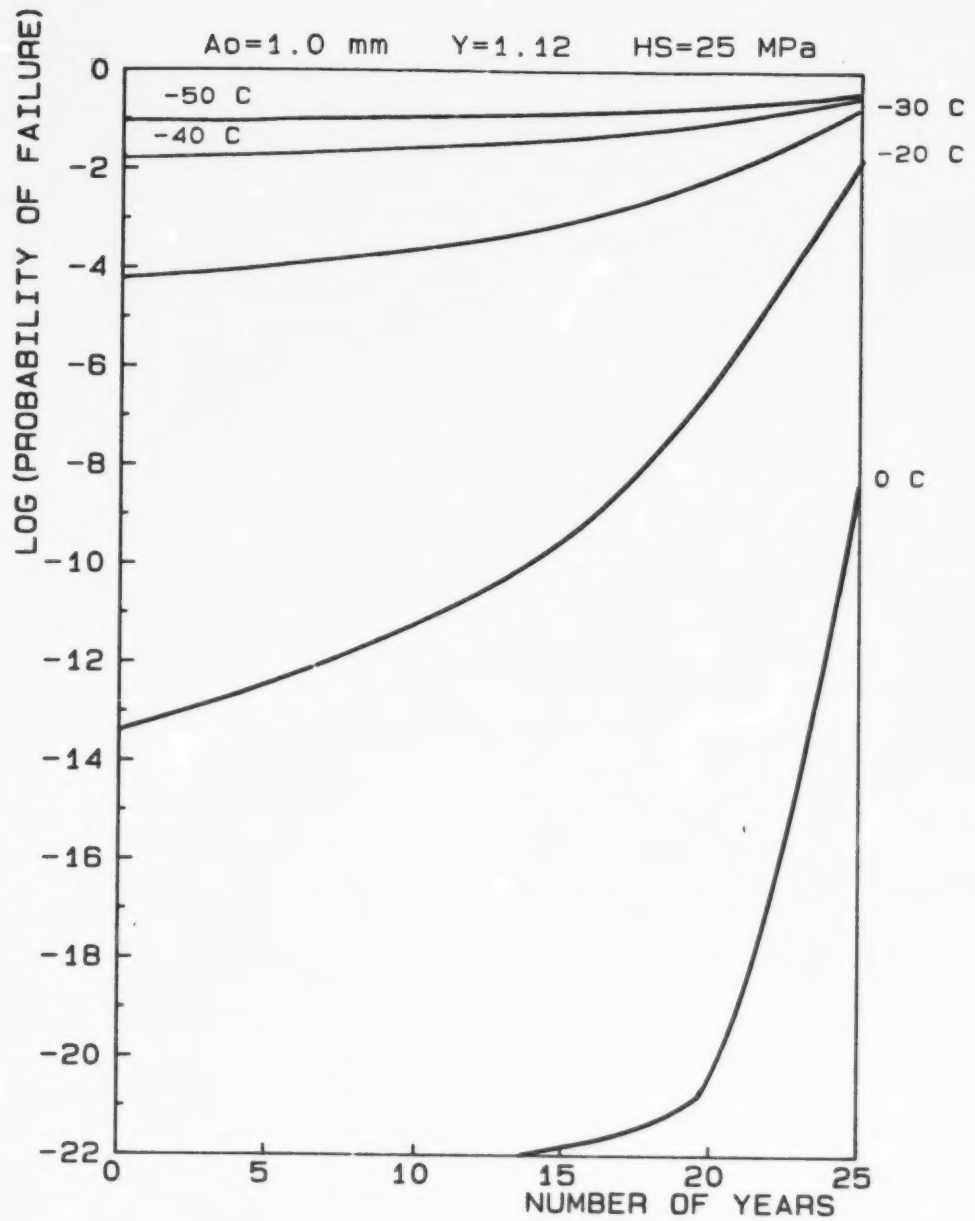


Figure C-74

- C71 -

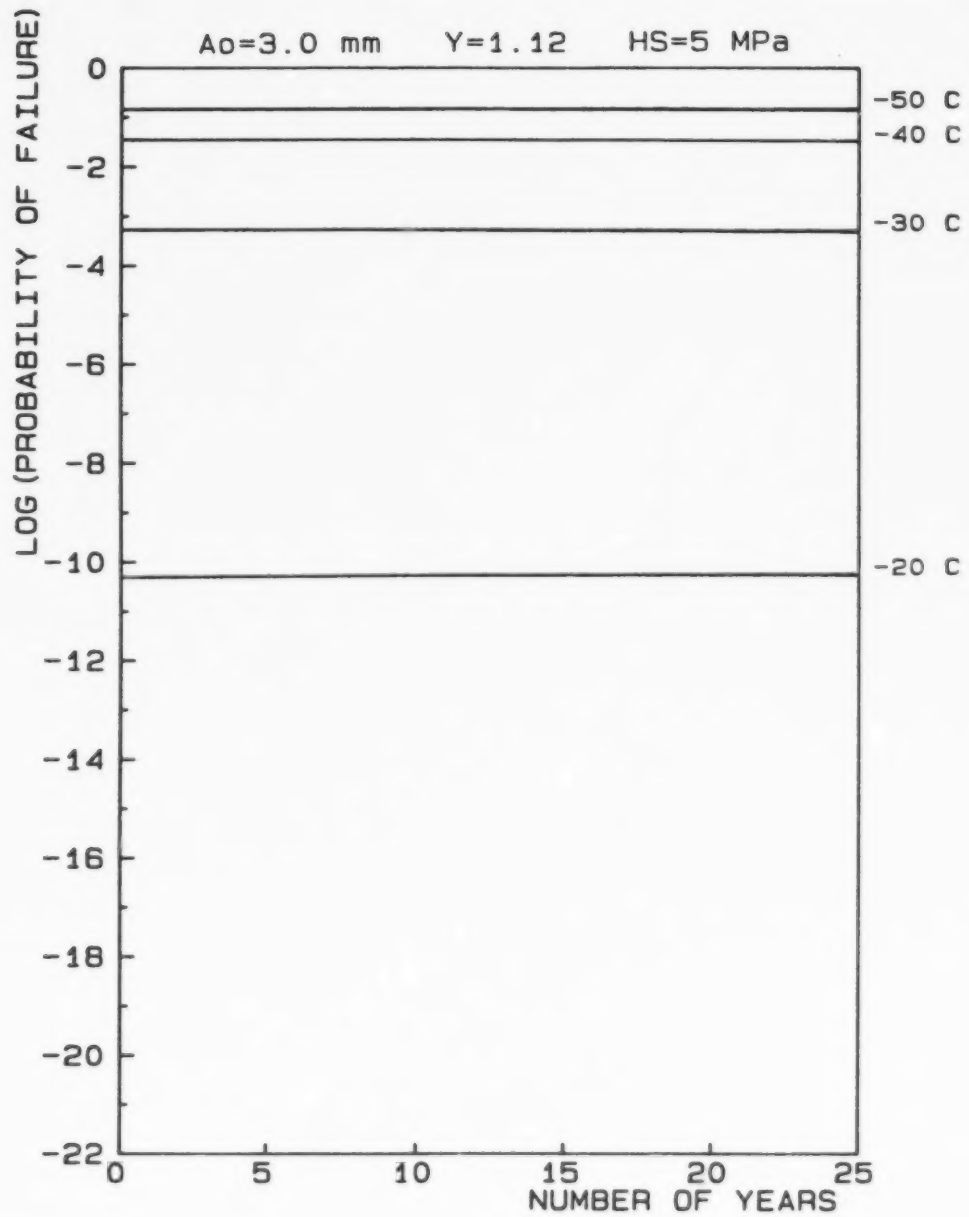


Figure C-75

- C72 -

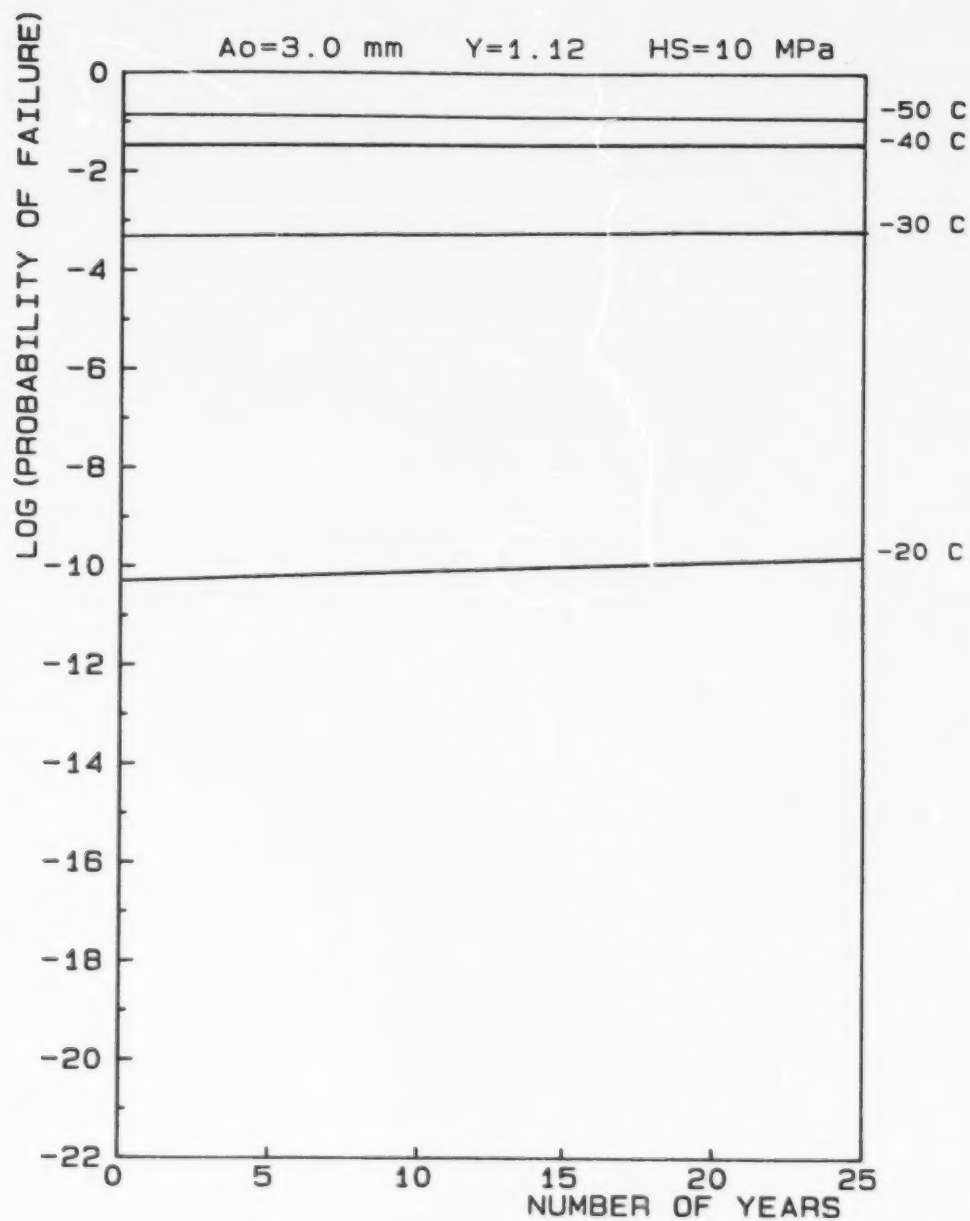


Figure C-76

- C73 -

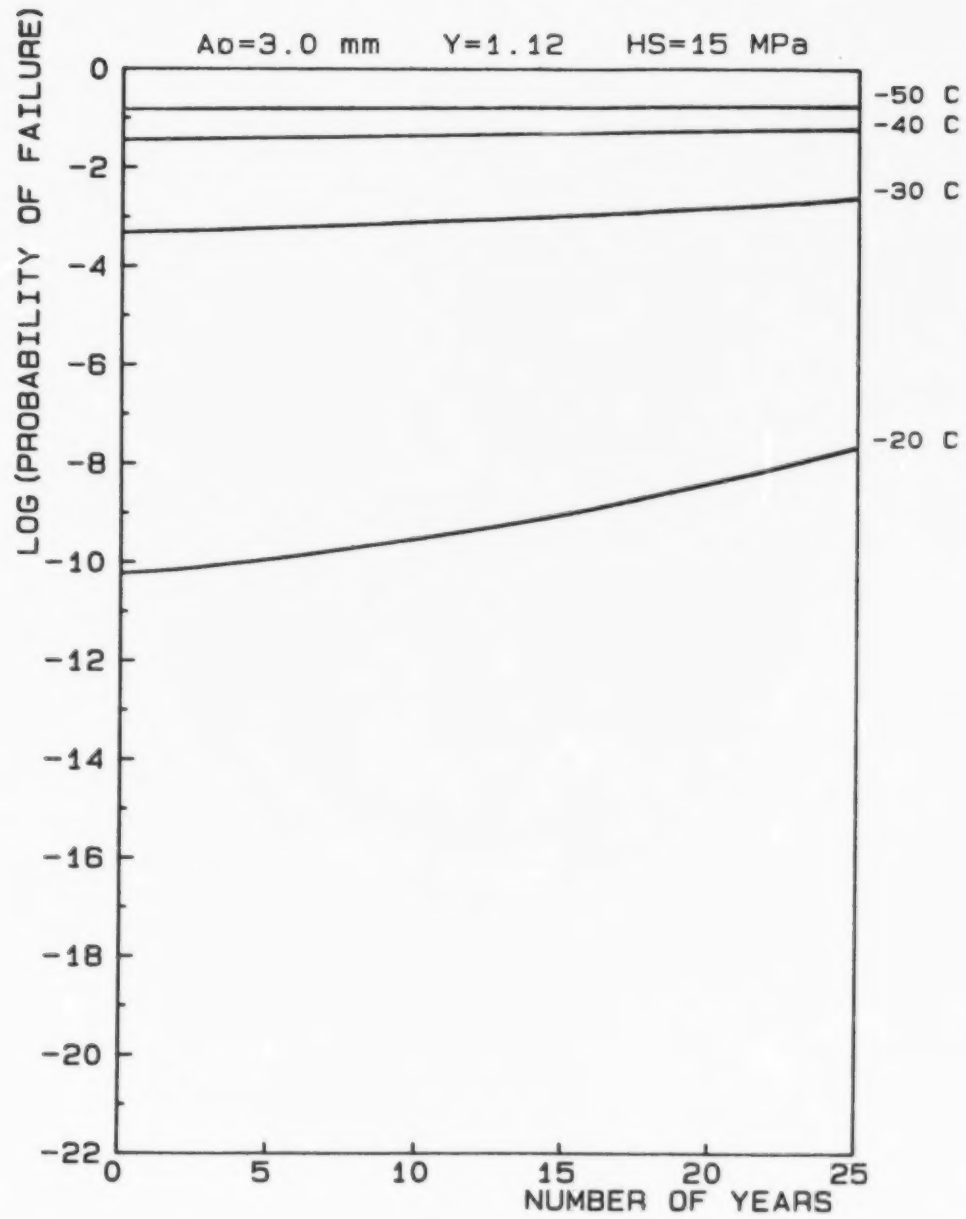


Figure C-77

- C74 -

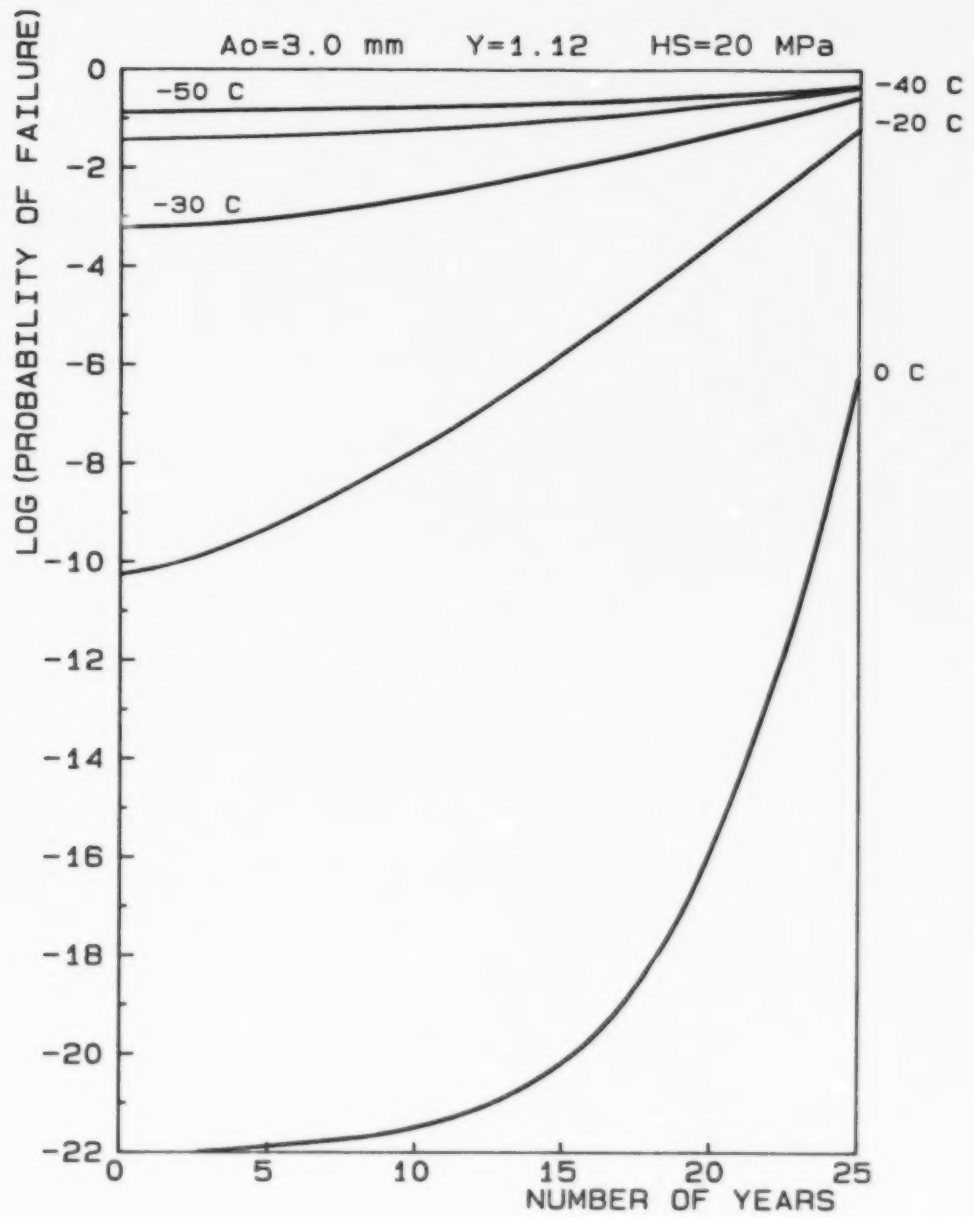


Figure C-78

- C75 -

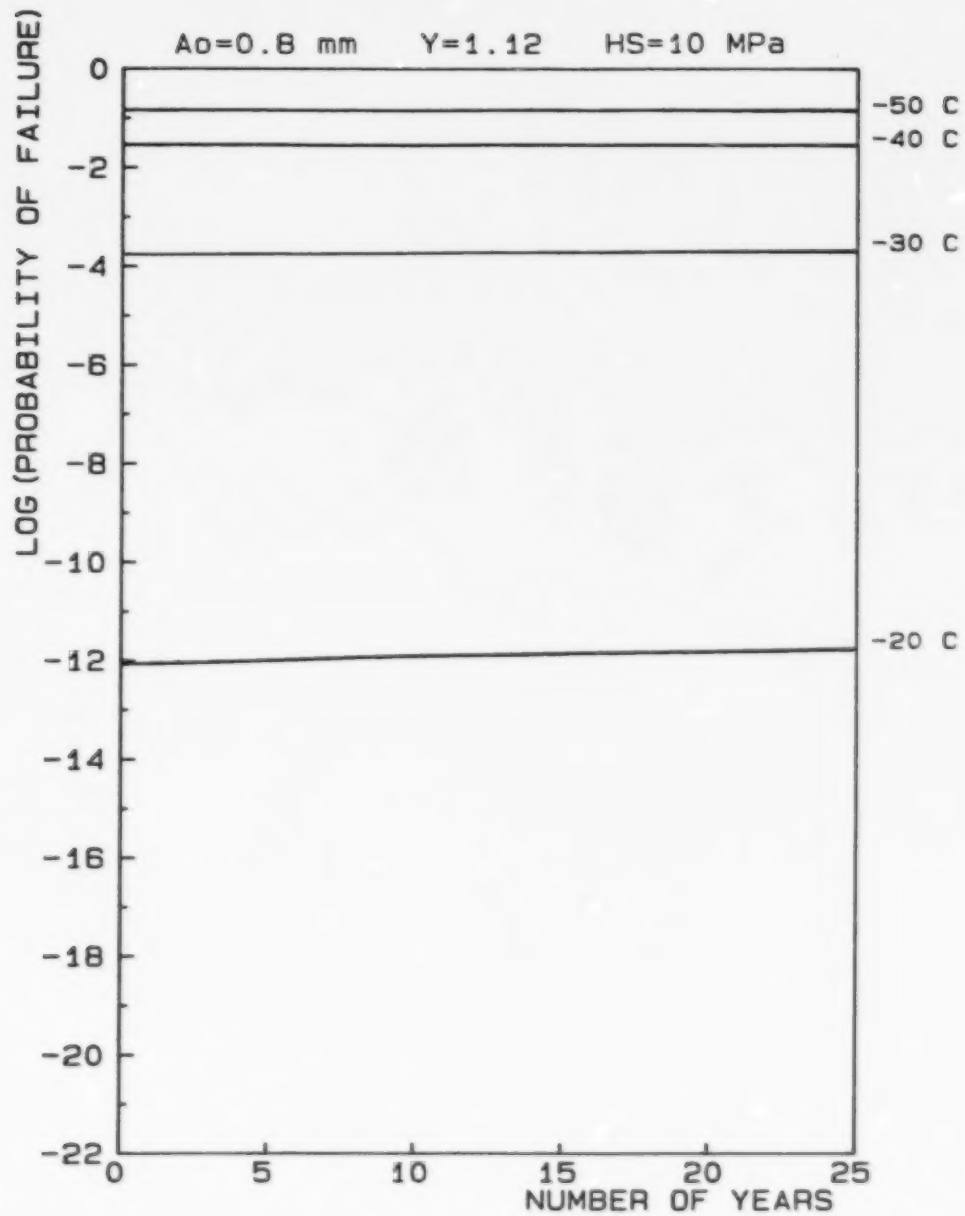


Figure C-79 to C-90

Probability of Failure as a Function of Time
K-Joint, $\Phi = 0.05$

- C76 -

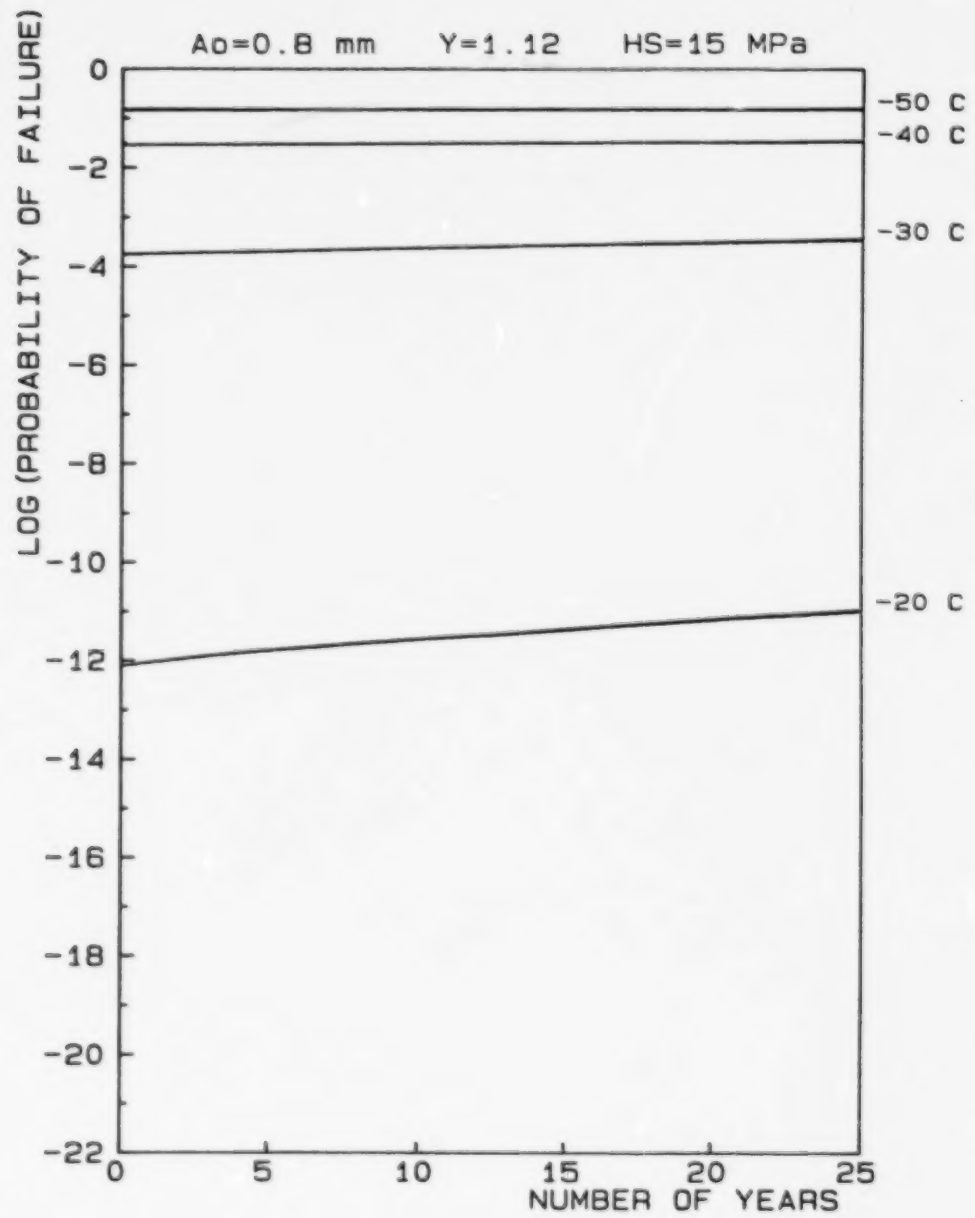


Figure C-80

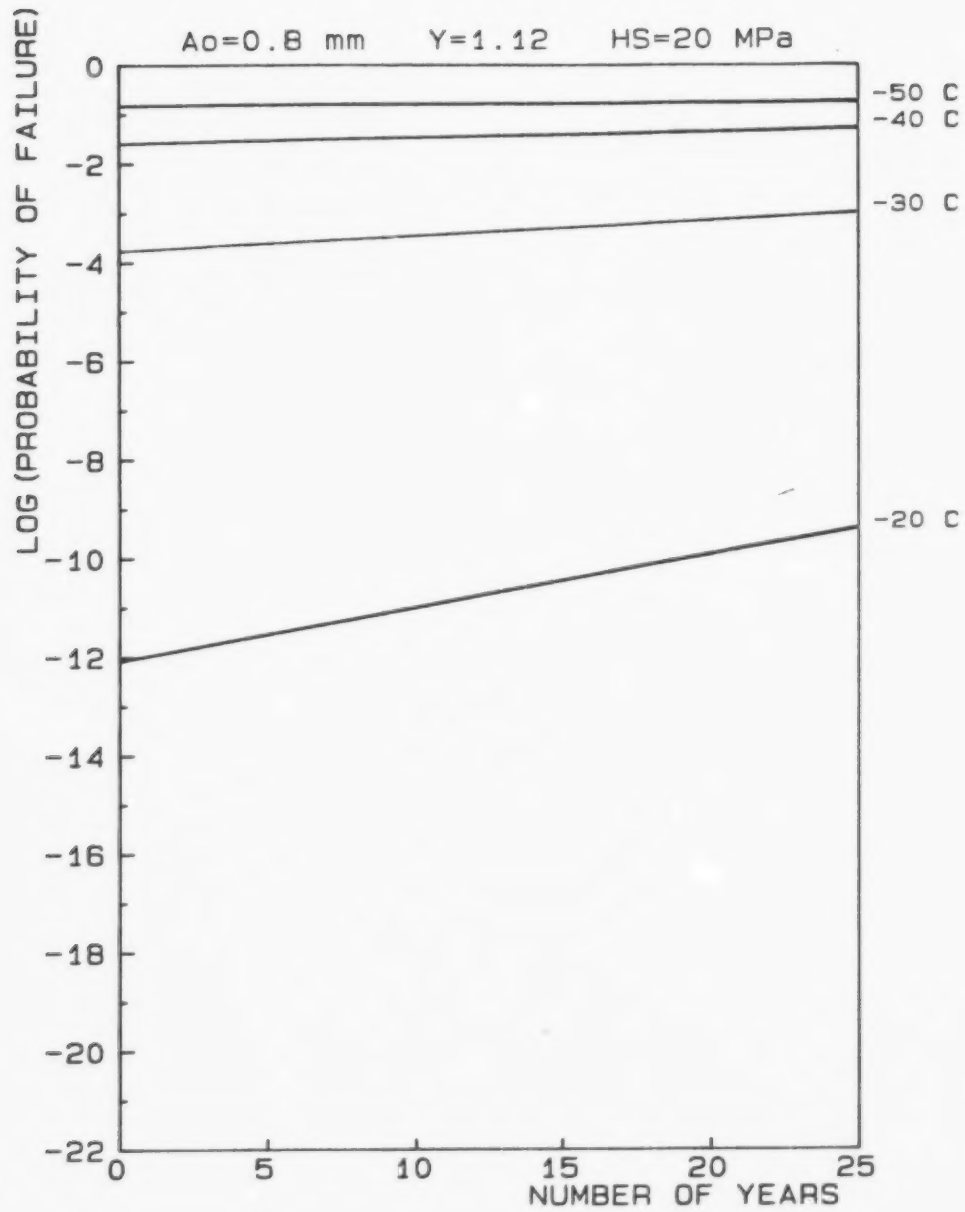


Figure C-81

- C78 -

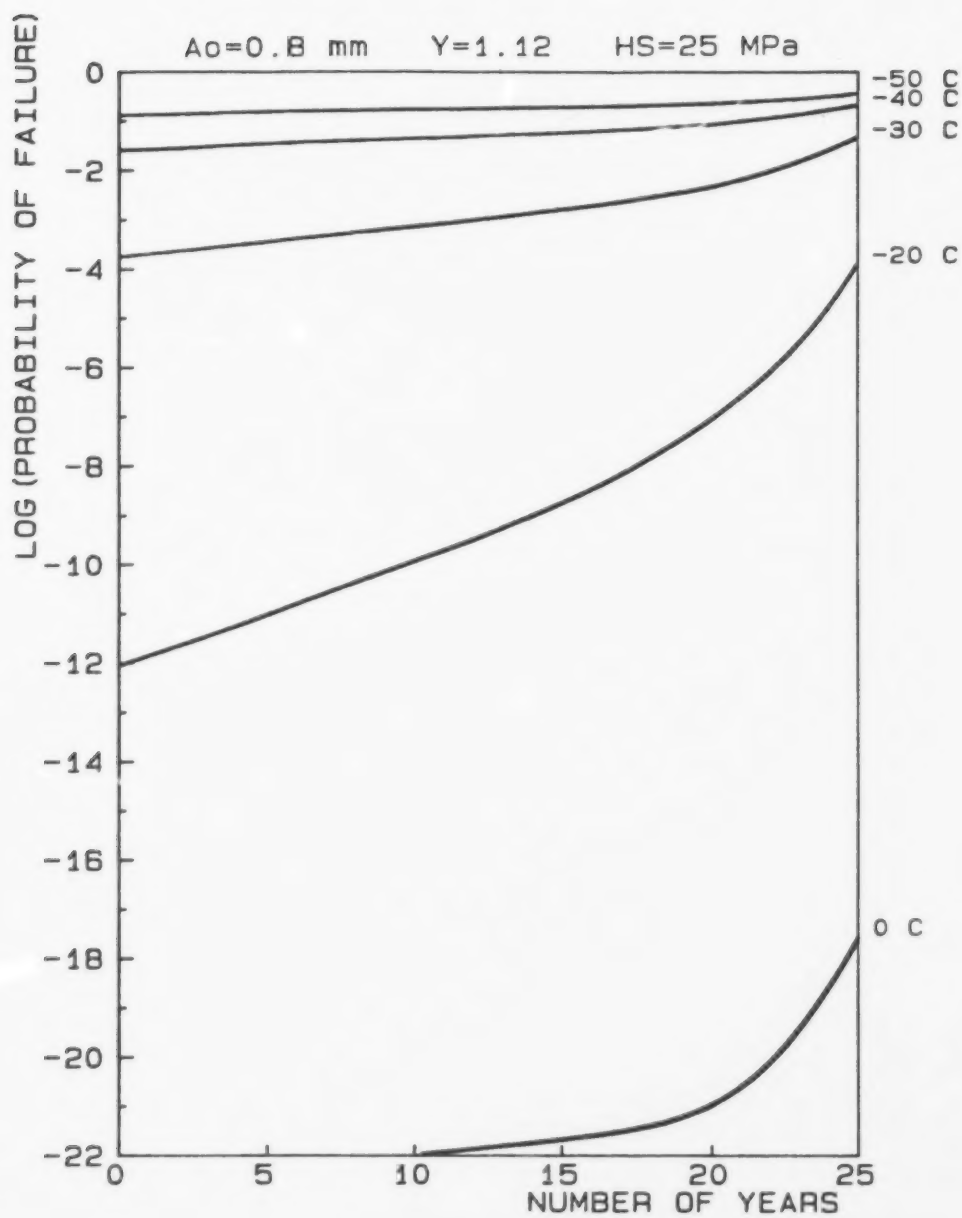


Figure C-82

- C79 -

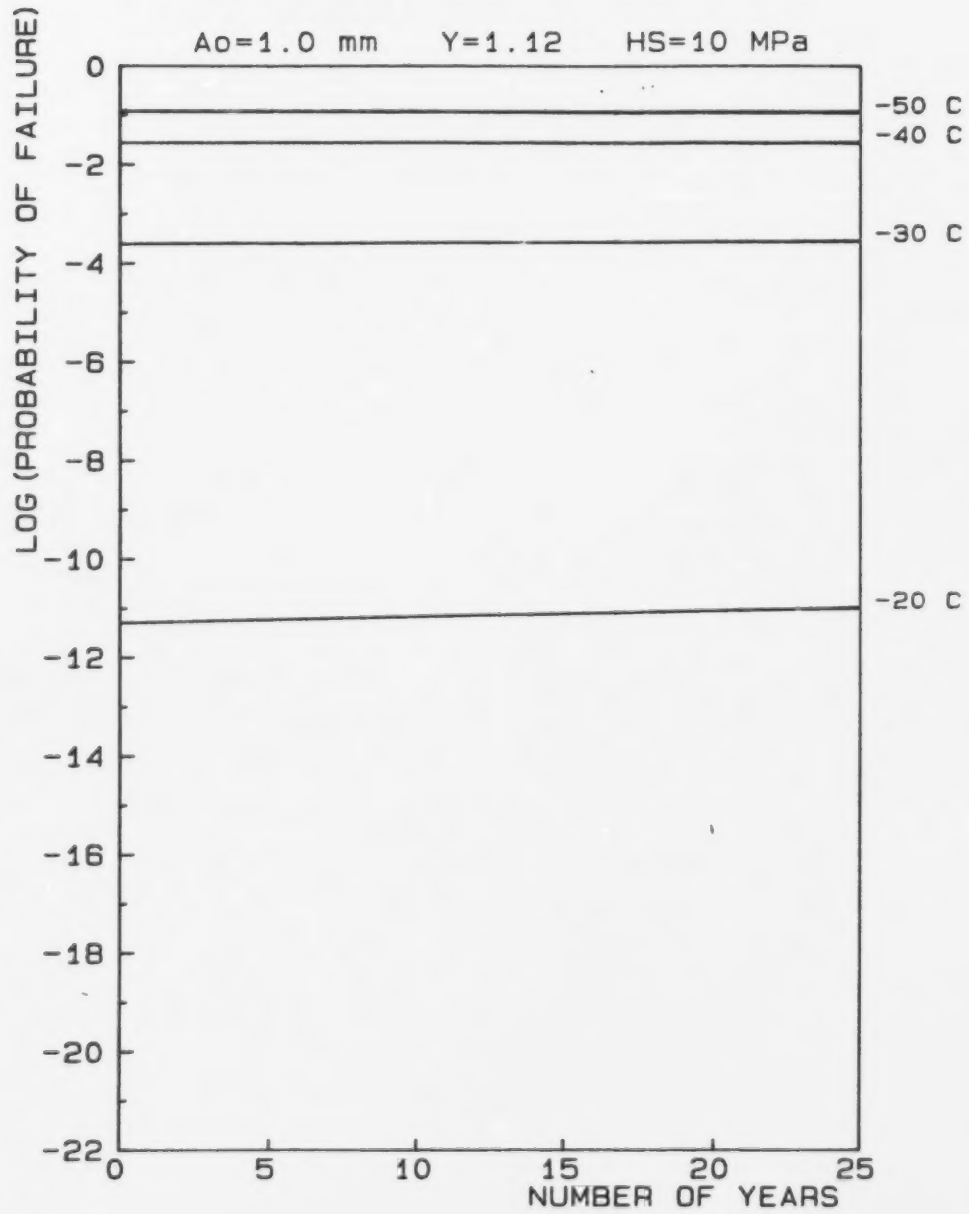


Figure C-83

- C80 -

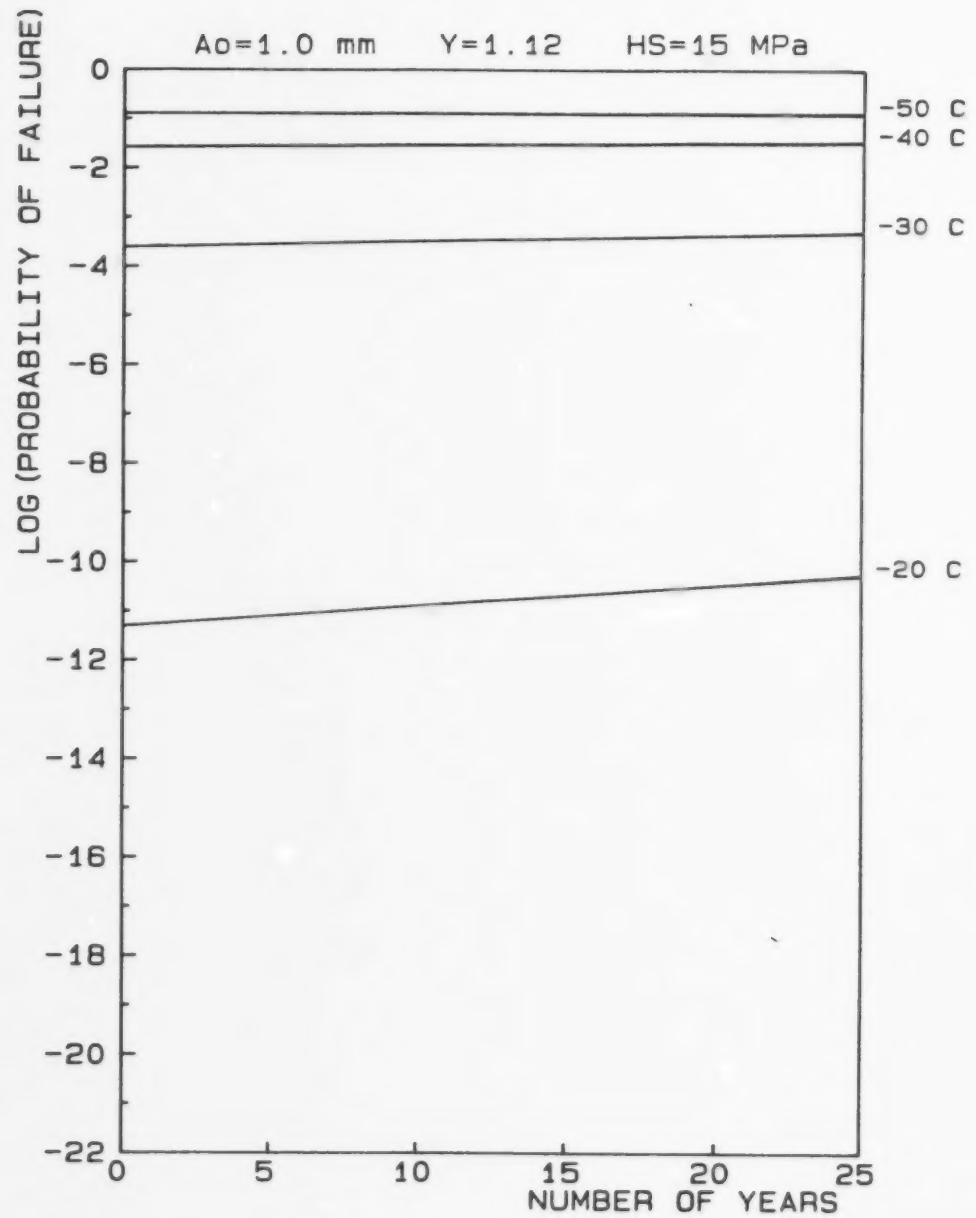


Figure C-84

- C81 -

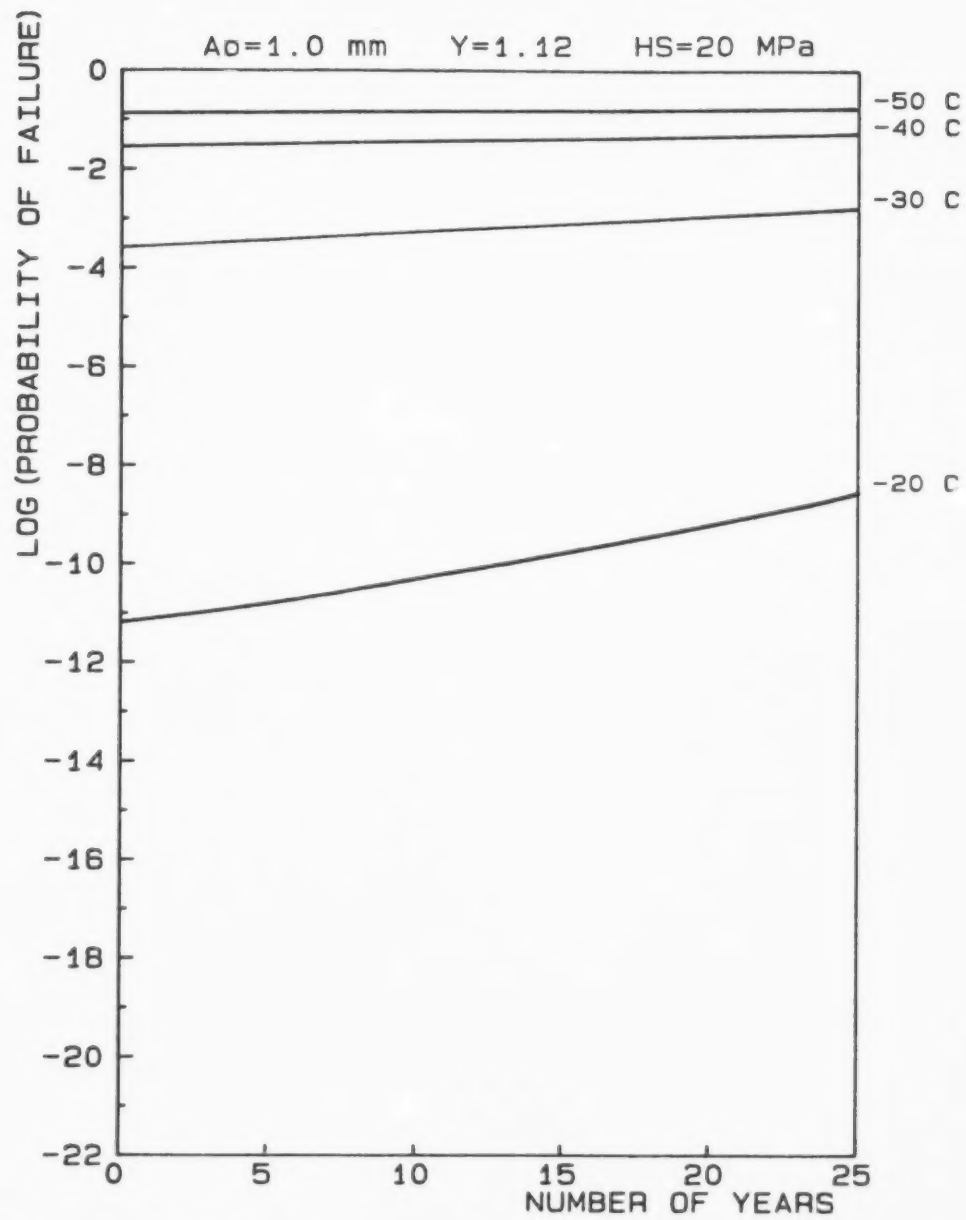


Figure C-85

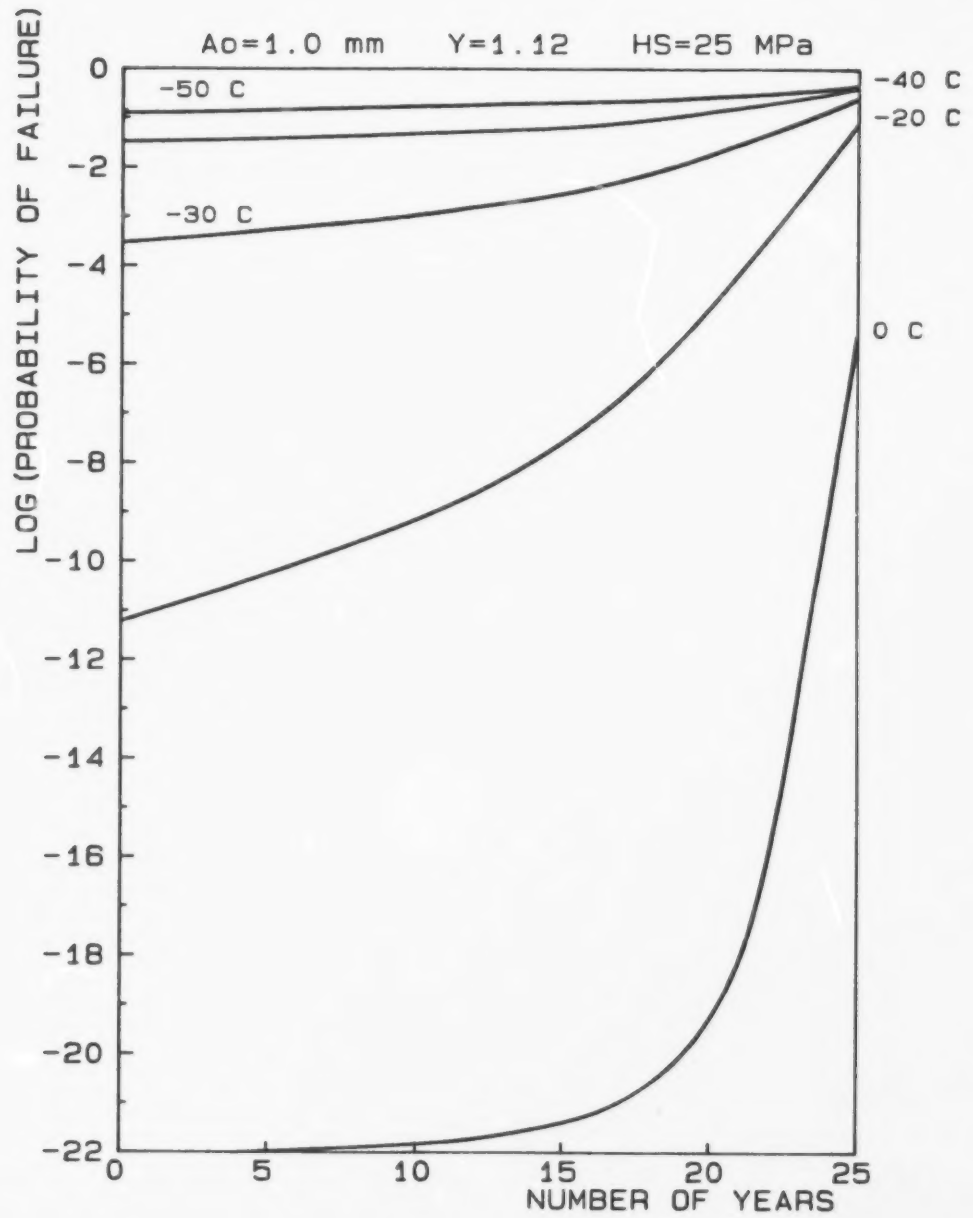


Figure C-86

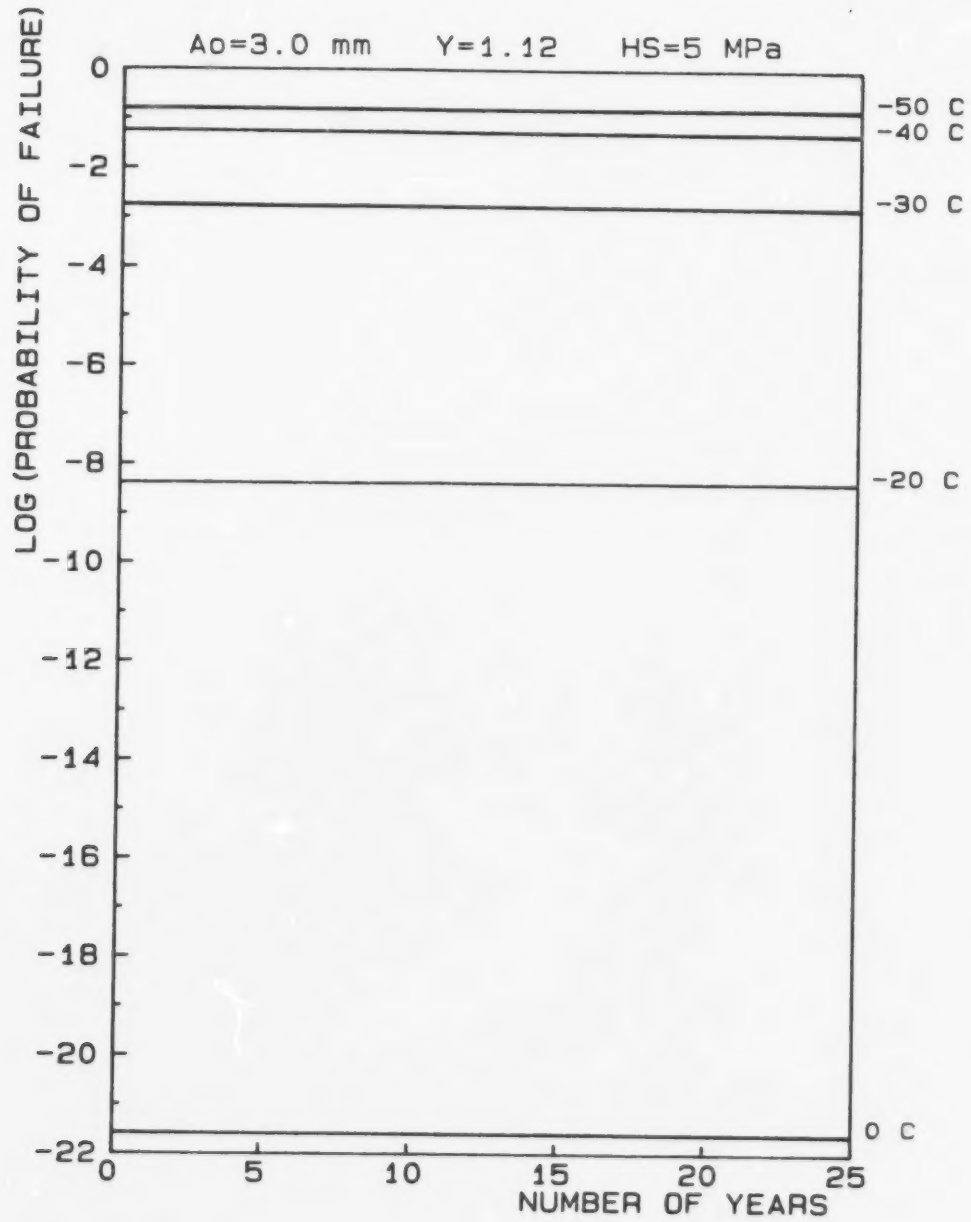


Figure C-87

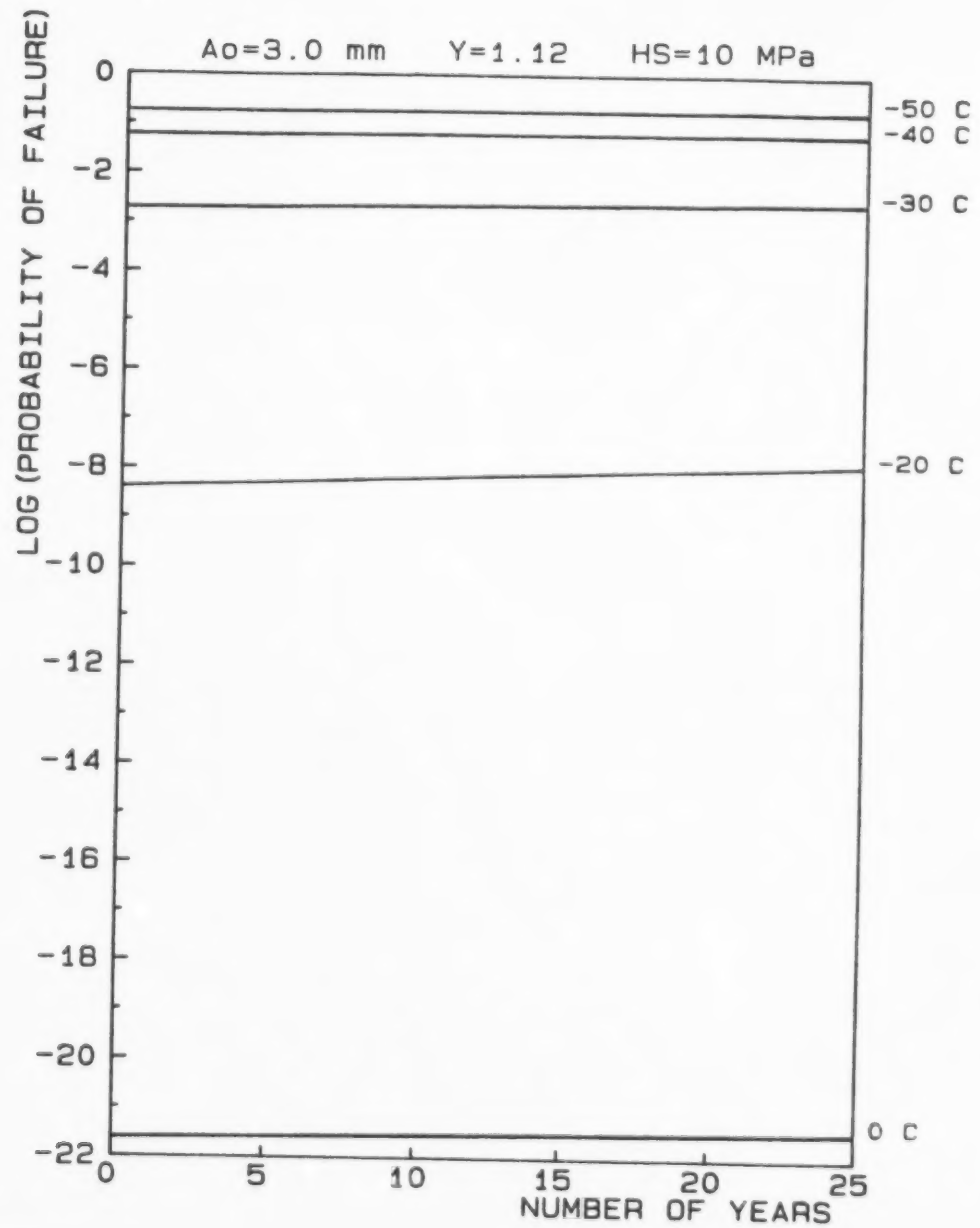


Figure C-88

- C85 -

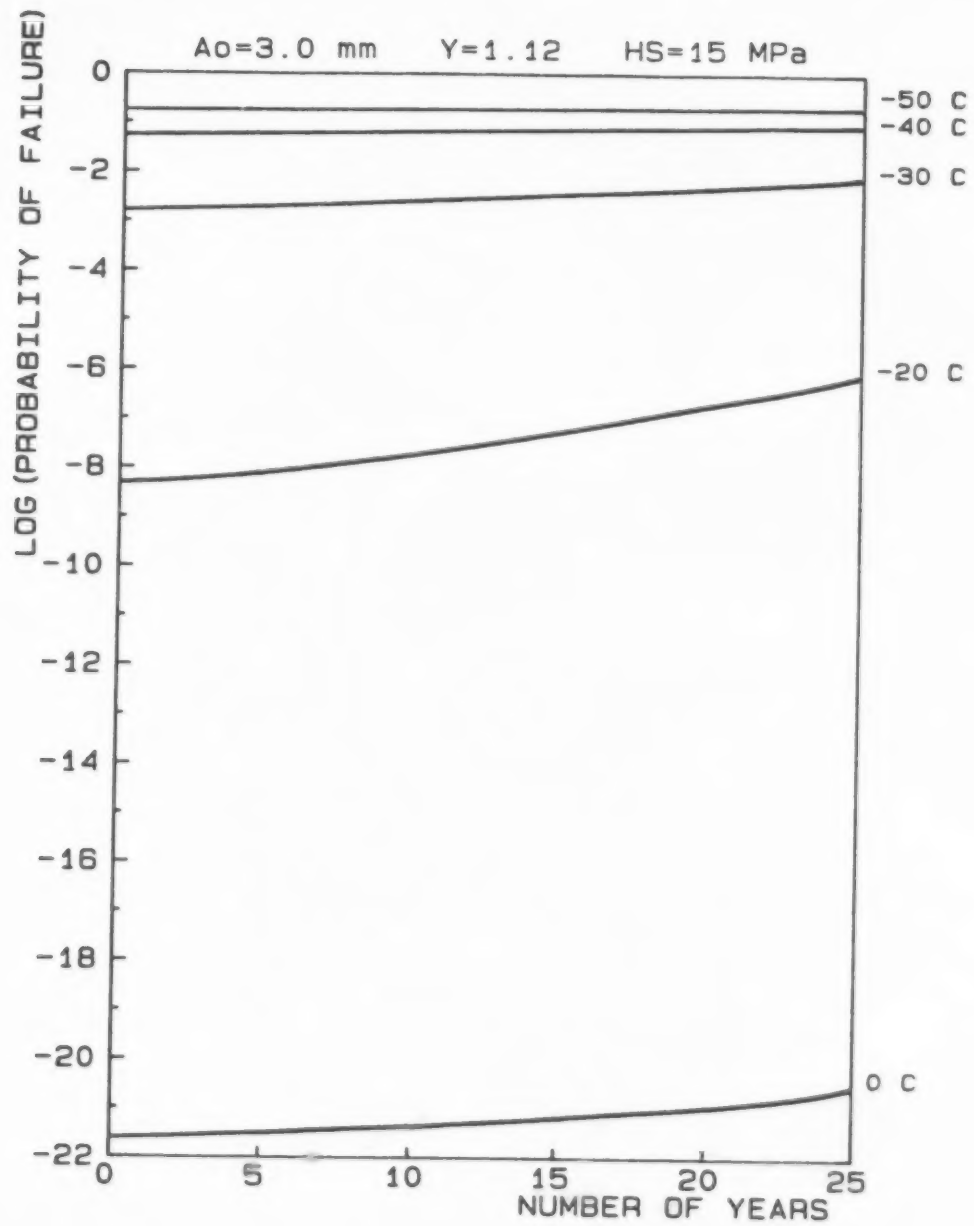


Figure C-89

- C86 -

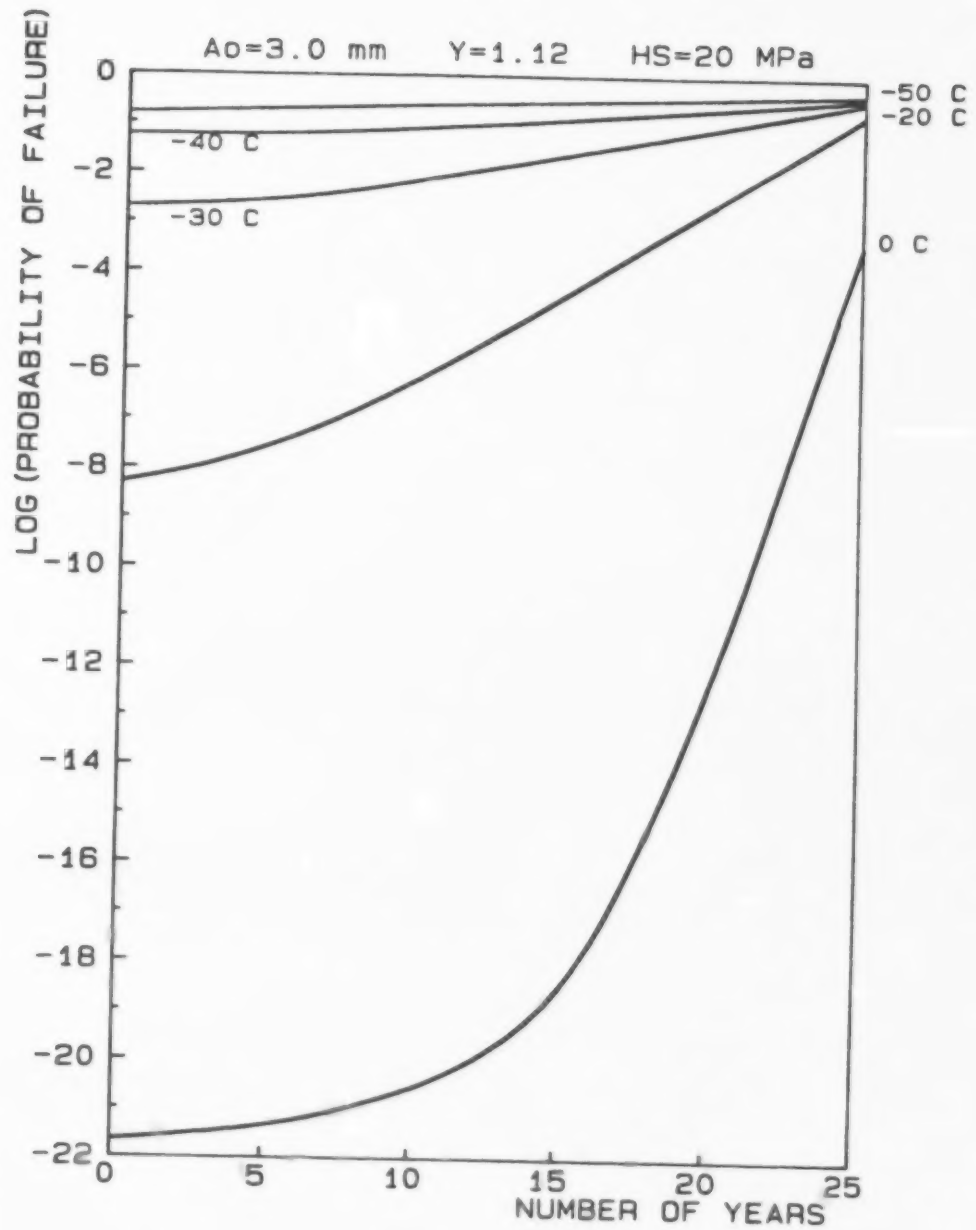


Figure C-90

- C87 -

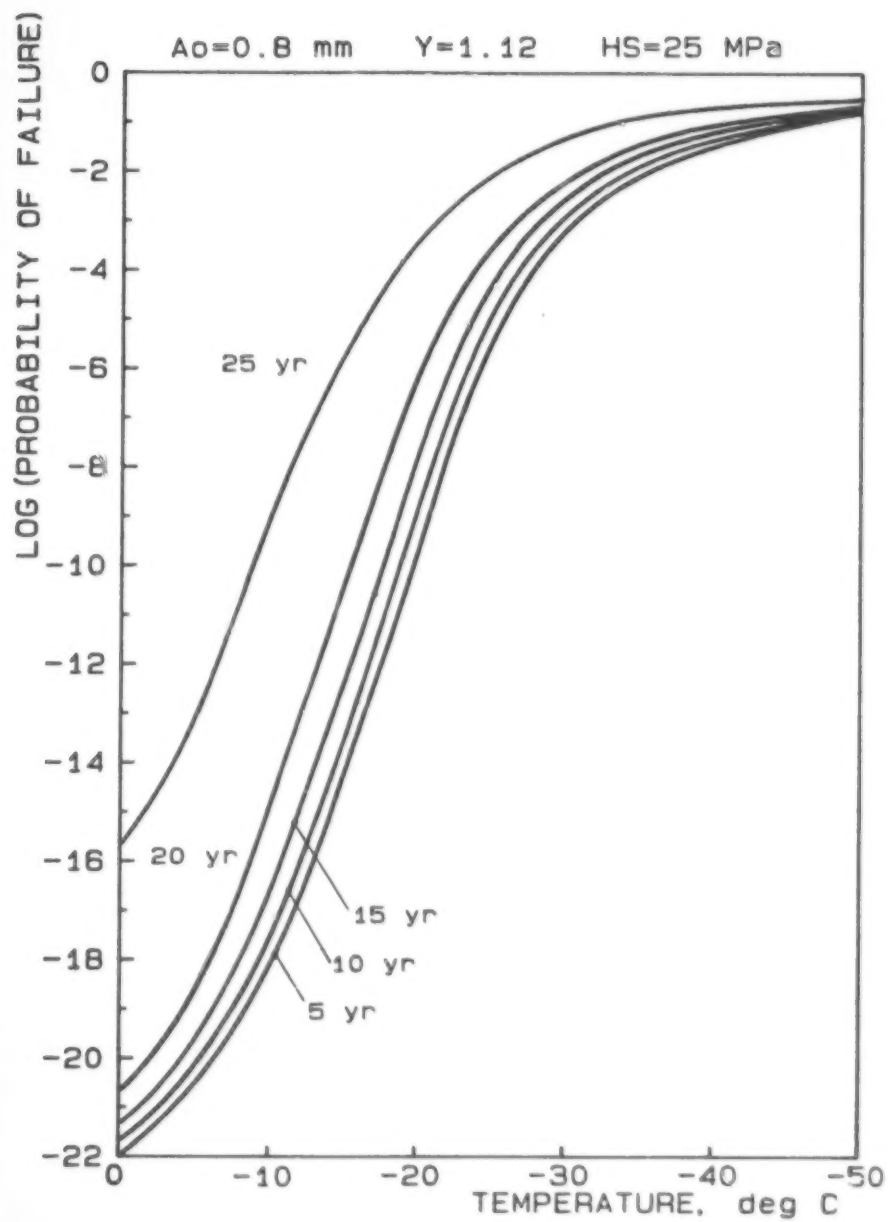


Figure C-91 to C-92

Probability of Failure as a Function of Temperature
T-Joint, $\Phi = 0.05$

- C88 -

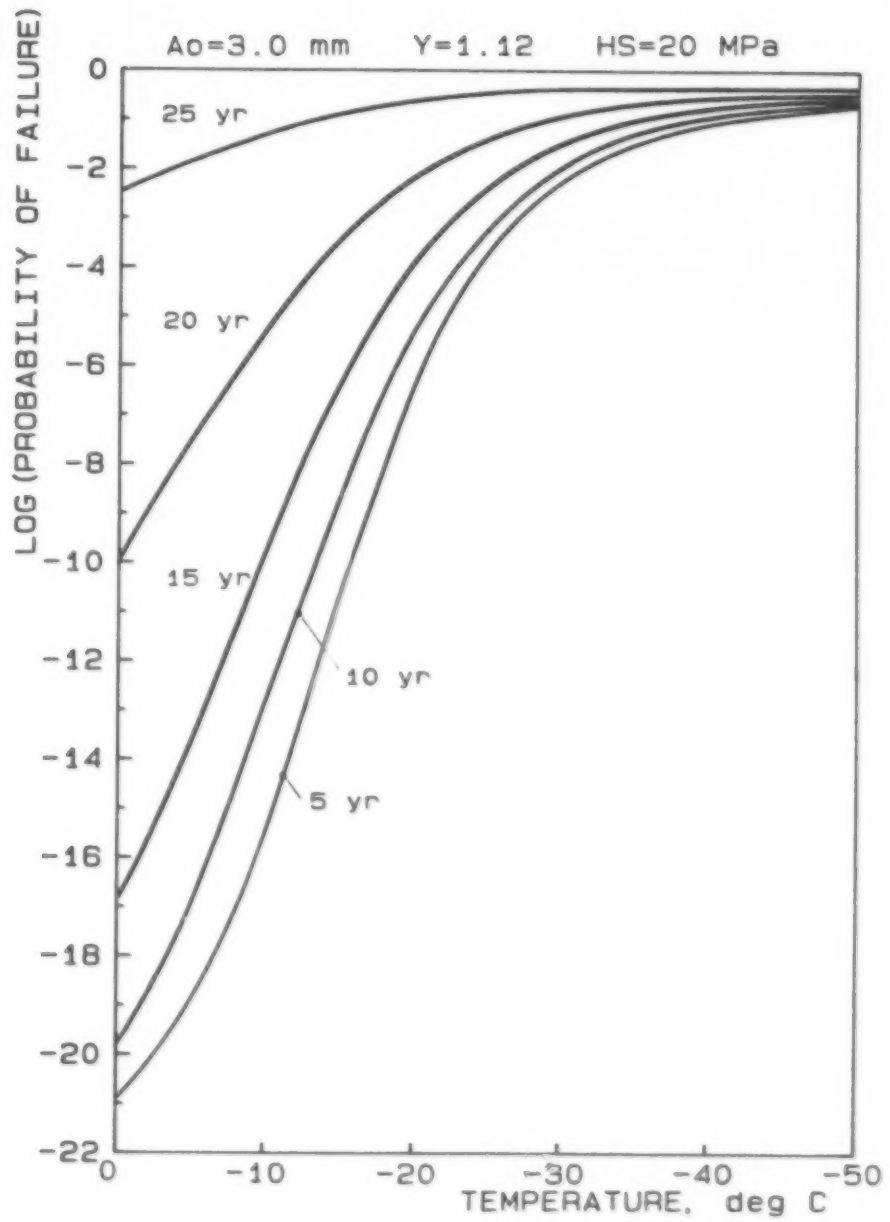


Figure C-92